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Electrolytic Theory of Corrosion

THE receipt recently of a copy of the "Journal" of the Society of Engineers, London, containing a paper on "Corrosion and Rusting of Iron" serves as a reminder that the electrolytic theory of corrosion has now received universal adoption—quite remarkable progress, for the theory was advanced less than a decade ago. It was to be expected that European scientists would take up the subject promptly and with their characteristic thoroughness investigate it from every angle and endeavor to find where it would break down. It has not broken down, but the testimony of investigators in Germany, France and England substantiates the argument put forth by Dr. Cushman in propounding the theory. It is worth while definitely to record the status of the theory—or explanation, since it is so generally accepted—for occasionally some one not well informed on the subject inquires as to its progress and the degree of its acceptance. There are not wanting those who, in their failure to grasp the theory or to investigate it, lightly cast it aside. The time has long since past when an engineer or scientist of standing can do this.

Overhauling Old Hypochlorite Plants

FEW purveyors of a domestic water supply at present have the audacity to deliver to consumers a water that they know is continuously polluted unless treated with hypochlorite. However, Zanesville, Ohio, according to E. H. Babbitt, in a recent paper presented to the Illinois Society of Engineers and Surveyors, serves up the highly polluted Muskingum River water in its original condition. Following the floods of last year, a temporary bleach-treatment plant was built, but it is stated that the plant has never been properly operated, and although the water is not generally used for drinking, the typhoid death rate in 1913 was in excess of 100 per 100,000 population. In some of the Ohio cities, especially in Dayton, following the floods an actual diminution of sickness occurred, so thoroughly impressed were the residents by the sanitary measures enforced after the flood. There is no reason why Zanesville should not have enjoyed a similar record. About a year ago this journal called attention to the foolishness of complaints against the use of bleach and advised waterworks officials to maintain their records clear of any typhoid entanglements by keeping the factor of safety well above unity. Evanston, to which the former editorial referred, had a mild epidemic of typhoid following a heavy lake storm in November, and some residents ascribed it to the failure of the temporary plant. Most of the temporary plants installed two or three years ago, when "hypo" plant installations were at their height, must now be ready for junking, and no pains or expense should be spared in making the permanent plants capable of taking full advantage of a thoroughly tested and approved method of controlling the sanitary quality of drinking water.

Supporting Large Pipes

LARGE pipes carried across streams and valleys offer a number of interesting problems. If the supporting piers are closely spaced, the masonry costs are high and the waterway, if over a stream, is restricted. If the spans are fairly long the reactions are excessive, and as a consequence of this and the beam action objectionable stresses are introduced. Many designers, therefore, use bridges to carry the lines; but this is generally an expensive solution of the problem. In steel pipes and spans of less than 400 ft., a self-supported arched conduit is possible, but the diameter must be large, so as to enable the pipe to take care of wind stresses, buckling, etc. In the case of wood-stave pipes, however, so far as this journal is aware, this scheme has never been used, though it seems possible with a short span. At Saxon Falls a 72-in. line is carried on piers spaced 26 ft. 7 in. on centers. The resulting reaction at each pier would have been about 55,000 lb. To avoid the deformation and stresses due to this reaction the designer placed two 15-in. I-beam stringers between the piers to support the pipe. The comparatively small reactions per linear foot of pipe along the line of contact with the stringers are taken care of by the pipe itself, without extra provision. The scheme is very simple and though often proposed has not, so far as known, been previously employed.

Grade-Separation Appropriations

THE time is at hand for the Legislature and the Governor of New York to provide or fail to provide funds to permit the resumption on an adequate scale of grade-crossing elimination throughout the State. For several years such appropriations as have been voted by the legislators have been vetoed by the Governors, with the result that the work is almost at a standstill. This year's Legislature has already made a bad start. The response to the request of the Public Service Commission for the First District for an appropriation of $1,500,000 has been the introduction of a bill providing $600,000 for the entire State. That the daily press, while condemning in a general way the deadly grade crossing, fails to appreciate the magnitude of the work of removing the danger spots, and the responsibility of the State in the matter, is shown by a brief article in a paper published in the second city in the State. This paper blandly announces that if the bill passes, "abolishment of all grade crossings will be the edict." The author of this choice bit of information seems to think that the bill is something unique. He apparently is not aware that such appropriations as this have been in order since 1897, when the grade-crossing law was passed, equitably dividing the expense among municipality, State and railroad, and that if that amount had been set aside each year since 1897 the work would now be only well started. He must have forgotten that in his city alone six or eight times that amount has been spent in abolishing some of the grade crossings on one railroad. It is a pity that there is not general realization of the fact that to get rid of all of these crossings is likely to cost the State at least fifty times $600,000, and that if the annual death toll is to be reduced ample public funds must be provided every year to allow the public-service commissions to carry out a systematic and uninterrupted program.

Hiding Their Light

MORE and more engineers and engineering colleges are appreciating the value of publicity—not that which consists in rushing into print on every controversial subject or enlarging into sensations new engineering discoveries, but plain statements of occurrences which should interest engineers and the public. For some years the State universities of the Middle West have, through technical bulletins and through their press bureaus, secured much publicity for the investigational work of their engineering colleges and engineering experiment stations. Much of this work has been done for manufacturers and contractors, and in conjunction with State bureaus and departments. The effect of this publicity was well shown by a recent occurrence in the council of the Alumni Association of Massachusetts Institute of Technology. The council voted to appoint a committee to consider possibilities of co-operation between the institute and the State of Massachusetts. It was argued that someof the Western colleges enter into State and city work much more extensively than does the institute into Massachusetts affairs. In reply a great number of instances of institute co-operation with the State and private interests were pointed out. The real secret for the views advanced probably lies in the fact that the Western institutions have studiously secured publicity for their work, while the Boston school has not. It has, in a measure, hid its light under a bushel. The incident teaches a lesson which other schools will be quick to grasp. There is a wide difference between the legitimate publicity here referred to and the wild seeking after notoriety which all self-respecting engineers decry. The committee which has been appointed might properly bring in as one of its recommendations a plan for organized publicity work.

The Prospects of Engineering

FROM the communications which have recently come to the Engineering Record it is evident that the engineering professions, and particularly its younger members, are uneasy regarding the outlook for their future. There is no concealing the fact that on the whole neither pay nor prospects have of late been particularly encouraging. Technical pursuits of all kinds are, as a whole, underpaid. They require no small degree of capacity, coupled with long training, and it is not encouraging under such circumstances to look forward to years of service with relatively small remuneration. Yet in comparing the rewards of the engineering professions with those of other professions the situation does not appear by any means as bad as it might be. Young men of keen mind and intellectual tastes for years crowded into the three so-called learned professions of theology, law, and medicine, just as within the last twenty years many of them have headed for engineering. Doubtless the average engineer feels that, whether in routine work or in private practice, his returns are not commensurate with the laboriousness of his training. On the other hand, while the public does not seem to be willing to pay overmuch for the conservation of its works of construction, it seems to value its souls at a still lower figure, for, however little the engineer may be paid, he has on the average rich compensation compared with that of the clergyman. The average man in the clerical profession, as a matter of cold-blooded statistics, gets less salary than the average motormen or conductors of similar length of service on a first-class electric railway, and the latter can count upon steady jobs if they are faithful and industrious, while the former may be thrown out into the cold by a churchly convulsion and find great difficulty in again getting a foothold. And on the whole the higher-paid engineers fare much better than the higher-paid clergymen.

In law the average earnings derived from strictly professional work are probably little, if any, higher than in engineering, although the exceptional lawyer unquestionably makes very much more than the exceptional engineer, or has done so up to the present time.

Medicine is notoriously a less well-paid profession than law, and particularly as regards the younger men, although those of high reputation can show large yearly earnings. In proportion to the total number of doctors there are probably more incomes over $10,000, let us say, than in the case of engineers, although the average in the two professions is probably not far from the same. One thinks offhand of some distinguished specialist whose office is crowded the year round, but forgets the great army of country practitioners who are on duty twenty-four hours a day in a sparsely settled country and find great difficulty in collecting their extremely modest fees. The average young engineer at the age of thirty, let us say, is faring quite as well as his brother in any of the older learned professions. The big prizes in engineering are comparatively scarce, but if one reads the signs of the times aright the outlook in this respect is steadily growing better. Mr. Craven's $20,000 per year as chief engineer of the New York subways is very exceptional as a salary in public service, but good salaries are more and more likely to come to competent men as the public realizes more and more the importance of their services from the standpoint of sheer economy, if nothing better. Still greater cause for encouragement is to be found in the demand for men of first-class engineering training in responsible executive positions. Mr. Waite, the city manager of Dayton, is a good example of the kind of fortune that is more and more likely to come the engineer's way. With the present trend of business such opportunities are sure to develop more frequently. The whole engineering profession, for example, should have rejoiced when one of their brotherhood found his way to the head of the biggest electrical company in the country. Every large enterprise in the future will have to depend on a high-grade of technical ability in executive positions. High finance is somewhat less pouplar than it was, and the plain stockholders are beginning to look about for men of tried ability to take the place of clever jugglers of securities. The spell is beginning to work, although slowly, and unless all signs go awry the next decade will see many big concerns getting down to first principles and depending on men who know how to do things superlatively well in their own lines of activity. Engineering, therefore, is going to look up instead of down, and the men who are now showing their capacity in minor positions have a better chance than ever of finding their reward.

Is Cancer Water-Borne?

THAT cancer is water-borne and will some day be included along with cholera, typhoid and a number of other germ diseases in the list with which the water analyst has to deal is suggested more strongly than ever before by the recent investigations on cancer in trout by Dr. Harvey R. Gaylord, director of the State Institute for Malignant Diseases at Buffalo, and by the U. S. Bureau of Fisheries. Every one of the nineteen hatcheries examined from Maine to Yakima, Wash., had cancer incidence, and the same thing is said to be true of hatcheries in all parts of the world. Curiously enough, only trout grown artificially have the disease. Turned loose, many of them recover. Virulence is greatest in the last of a succession of ponds on the same stream. Dogs drinking the water develop the growths, and so do rats fed on the scrapings from the hatchery troughs. Still the mystery is unsolved because one of the established facts about cancer is its non-contagious nature. "World's Work" for February, in commenting on Dr. Gaylord's work, calls attention to the possibilities of even more intricate means of communicating disease than the man-mosquito-man route of yellow-fever transmission. Transmitting cancer by means of water is not a new theory, but what are the details of the transmission? Is there an intermediary host—bacteria which an infinitesimal cancer bacterium infects? Perhaps the chemical residuums of life and the very care which domestication involves tend to reduce, as in humans, that power of producing sufficient opsonin or resisting force to the attacks of the numerous disease germs to which the human economy is continually subjected. At any rate, the subject is of such vital interest that analysts will no doubt be glad to co-operate with those already in the field searching for preventive means of transmitting cancer. Possibly some one has already tried filtering and chlorinating the water, carrying cancer to the last pond in the series noted. Should this or any other chemical water treatment prove beneficial the fact would be of tremendous import to the waterworks field.

Tendencies in Electric Pumping-Station Design

IN the design of electrically operated pumping plants local conditions determine the type, size and general arrangement of equipment, the provision for future development and facilities for maintenance. Beyond these features, however, certain refinements in practice peculiarly suited to electrical service may profitably be considered. They bear closely upon flexibility of control and reliability of operation, and are well illustrated in the general plans lately completed for a motor-driven fire service pumping station to be built in Boston as an auxiliary to the city's high-pressure water supply system, in the design of the electrically operated lock system at the Charles River Dam in the same city, and in other installations.

The use of motor control equipment operated from a common center some distance away from the pumps, the provision of automatic limit switches and special connections between motors and valves interlocking their operation, employment of pilot lamps showing the operating conditions prevailing, subdivision of bus bars and arrangement of emergency feeders are essential features of a high-class installation in cases where a large degree of flexibility in service, ease of control and reliability at all times are demanded. In a small motor-driven pumping station for rural or town service it is obvious that the refinements of the large city plant are often superfluous or too costly to be provided; but the study of tendencies in practice in the larger field is none the less worth while, since individual features frequently can be adopted to local advantage.

In the concrete example provided by the Boston fire service pumping station reliability of operation will be greatly enhanced by a dual electrical supply, consisting of 60-cycle, 6600-volt service from the Boston Edison Company, and 25-cycle, 13,200-volt service available in emergencies from the Boston Elevated Railway Company's lines. Additional security will result from the use of three independent Edison high-voltage feeders, and all auxiliary apparatus, such as lighting equipment and motors, operating valves and variable resistances, oil switches and signal lamps will be supplied from the 113-226-volt, direct-current underground distribution system of the Edison company, which is provided with a storage battery reserve of large capacity, insuring the most reliable service possible to a central station. On account of the difference in electrical frequency, each pumping unit will be equipped with two induction motors, either of which will handle the service, and as the motors will be wound for full line voltage, the station design will be relatively simple and additional space will be available for the interesting control equipment required.

Without going into the details of the electrical equipment, it is worth noting that the high-voltage bus bars will be so sectionalized that each section can be operated singly or collectively as a common bus taking current from one or more lines, as may be desired; that the connections will make it impossible for current to be thrown upon both motors of a pumping unit at the same time; that the 60-cycle and 25-cycle systems will each be provided with a separate control board; that the discharge valves will be arranged to open automatically when the motors attain the proper speed, the point of opening being adjustable, and that pilot lamps will indicate the position of all switches of fundamental importance in motor and circuit control.

Another feature is the provision of relief valves for pressure regulation which may be adjusted either in place or from the electrical control board, and the automatic acceleration of the motors and pumps and the assumption by the latter of their proper loads within about half a minute of the closing of the control switches will insure prompt service when it is required. In common with usual practice, bus and oil-switch compartments, cable terminals appropriately cared for mechanically and from the insulation standpoint, and separate quarters for switching appliances will be provided.

These features are to a large degree adaptable to general pumping station practice and illustrate the importance of mechanical and electrical problems in this branch of hydraulic work. It is doubtless true that freedom from hand manipulation carries some penalties in the line of increased maintenance cost, and remote control apparatus is apt to be expensive to install on a large scale, but just as the large central station cannot afford to be without many refinements of equipment which would be beyond the means of the small plant, the high-powered municipal pumping station planned for fire protection in a city of the first order rightly deserves the most complete apparatus available. Speed of starting operation, economical pump and motor sizes, a wide range of adjustment of output, and subdivision of service with a minimum amount of manual handling are desirable features which go far toward justifying elaboration of detail within the limits of dependable equipment design and arrangement.

Industrial Accident Analyses in Massachusetts

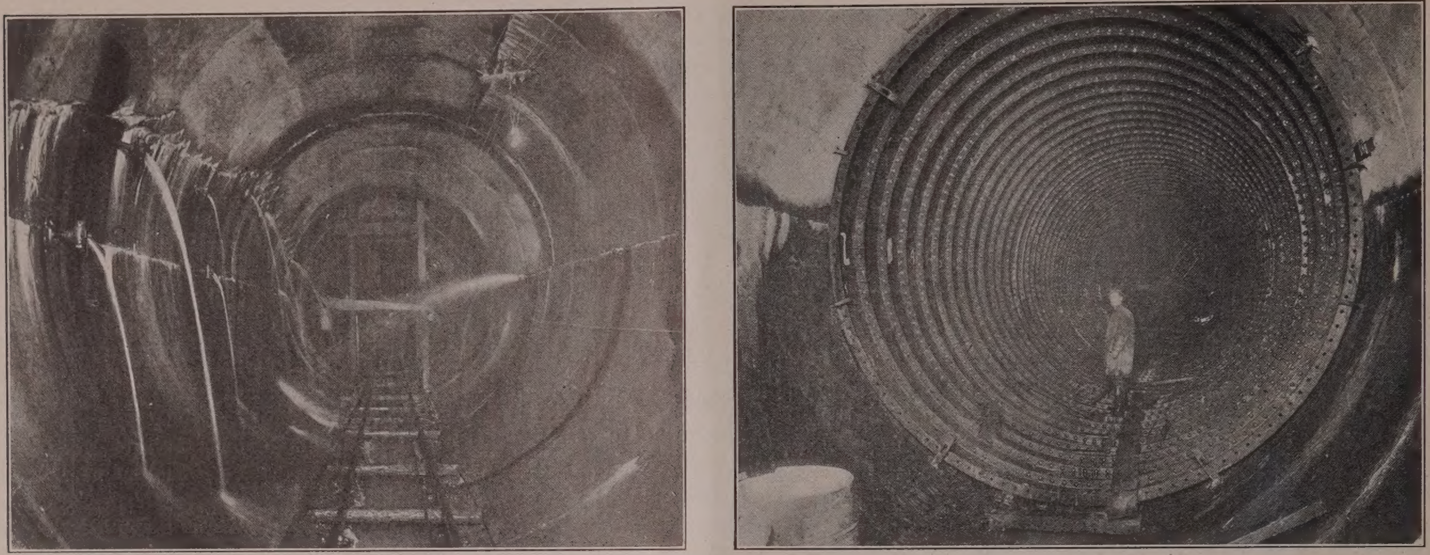
THE forthcoming first annual report of the Massachusetts Industrial Accident Board is likely to be in great demand when the richness of its contents from the analytical standpoint is generally known. The experience of the board in the administration of the workmen's compensation act of the State is of interest to all employers of labor because of the comprehensive manner in which it is set forth, the comparisons made with the practice of other States, and the suggested lines of improvement; while not a little engineering interest attaches itself to the analytical studies which the board has made of casualties in different industries, working conditions, hours of occurrence, wage scales and other economic factors.

Only brief mention can be made in these columns of the data with which the report is replete, but it is evident that human conservation is greatly needed not only in the more hazardous occupations but in trades ordinarily regarded as free from special risk. At least 800,000 wage earners in Massachusetts are eligible to the benefits of workmen's compensation. In the year ended June 30, 1913, there were reported to the board 474 fatal and 89,694 non-fatal accidents; the total number of days' work lost was 1,156,787; the average duration of disability was 12.89 days, and the average cost per accident reported, not including the cost of insurance administration, was $18.70. In other words, the consumer paid for every $10 worth of product purchased slightly less than 1 cent as the per capita cost for the actual losses paid under the compensation act. The unit cost is small, but the percentage of workers yearly involved in injuries is clearly too large to be tolerated without comprehensive efforts in the direction of preventive surveys of plants and recommendations having power of enforcement back of them. These are bound to come, and only the restricted functions of the board have held back this most important work. Through the recent legislative creation of a joint commission, consisting of the Board of Labor and Industries and the Industrial Accident Board, the power to order the installation of safety devices and to make regulations covering occupational diseases, together with the maintenance of a museum of safety, are certain to improve present conditions materially.

Before the compensation act went into effect the workmen of Massachusetts netted less than $400,000 yearly in payments under employers' liability insurance, while in the year covered by the report $867,644 was paid for non-fatal accidents, the total medical cost being $414,195 and the compensation to dependents of injured workmen $395,542. The average weekly payments in compensation were between $5 and $6. The board is of the opinion, however, that several far-reaching amendments are advisable, one of which would provide a more adequate schedule of compensation values and another which would specifically extend the scope of the act to occupational diseases. It believes that the cost of insurance can be reduced one-half by a campaign of education and legal enforcement for securing healthful working conditions and the adoption of proper safeguards. During the year the employers paid $4,000,000 in premiums to the insurance companies, and to eliminate needless expense in this direction the board urges stricter insurance regulation and expresses the opinion that State administration of industrial insurance will eventually come to pass. The board feels that an average charge of 1 per cent of the payroll should be sufficient to maintain workmen's compensation under well controlled conditions. Increases in the payments and in their duration, and particularly the more scientific determination of schedules with relation to the occupation, injury sustained and other factors, are deemed especially desirable. Other improvements advocated are additional compensation for permanent incapacity, payment of lump sums in special cases, the establishment of branch offices of the board, supply of full information to the commission, a larger latitude with regard to the employment of physicians, and broader applications of the arbitration principle.

The detailed study of accidents by the board includes numerous tables and graphic analyses of conditions helping to throw light upon hazards in classified occupations. The largest number of accidents was reported from the iron and steel industry, the total for the year being 13,568, with only 6 less in the textile field. In road, street and bridge transportation 11,398 accidents occurred; in the building trades, 7310. In fifteen manufacturing industries, including electric-light and power companies, there were 10,698 accidents. The largest number of fatalities occurred in transportation. Thirty-two per cent of the total number of accidents was caused by hand labor, 12 per cent by machinery peculiar to special industries, 9 per cent to falls, 4 per cent were eye injuries, and 0.1 per cent were occupational diseases. The accident peaks were between 10 and 11 o'clock in the morning and 3 and 4 in the afternoon.

Other data as to accident frequency, earnings of victims, ages, periods of special injuries in relation to withdrawal from work, and various vital statistics add to the value of the report and illustrate the opportunities for research in this new branch of industrial economics.



Leakage into Rondout Tunnel after Hydrostatic Test and Rings for Reinforcing Ruptured Section

Reinforcing Ruptured Concrete Lining in High-Pressure Aqueduct Tunnel

Repairs to Rondout Siphon of Catskill Aqueduct Involved Use of 730 Tons of Electrically Welded Steel Rings Erected in Cramped Quarters

By JOHN P. HOGAN
Division Engineer, Board of Water Supply, New York

ELECTRICALLY welded steel rings, aggregating in weight 730 tons, have been placed within the Rondout pressure tunnel of the Catskill Aqueduct at High Falls, N. Y., to reinforce three sections of the 14 ft. 6 in. circular concrete-lined tube which were ruptured during a recent hydrostatic test of the completed conduit. The tunnel is 23,608 ft. long and was driven through rock from eight shafts ranging in depth from 374 to 710 ft. The average thickness of the original concrete lining was 20 in. Work was completed in July, 1912, and when the tunnel had been partly filled to a level 58 ft. below the hydraulic gradient with water to test its tightness a leakage of 2600 gal. per minute developed. The indications were that this leakage, under actual service conditions, would amount to 3500 gal. per minute, or 5,000,000 gal. daily. The tube was immediately unwatered and cracks in the lining were discovered near shaft 4 opposite mud seams which had been encountered during the driving of the bore. The location of the mud seams is shown in one of the drawings.

Reduction of Leakage

The main body of the concrete lining appeared to be intact and where ruptured the evidence was that the upper half of the tunnel had simply been lifted gently off the lower half by the heavy hydrostatic pressure. It was then decided to strengthen the weak portions of the tube by welded rings of 15-in. steel channels incased in concrete. These repairs, reducing the tunnel diameter from 14 ft. 6 in. to 12 ft. 9 in., have reduced the leakage to 850 gal. per minute. Each ring weighed 2430 lb. before welding, and the total weight of steel erected and riveted was a little more than 1 ton per foot. There were 1824 welds made, 73,000 rivets driven and 24,000 linear feet of calking done on flange joints between rings.

Articles describing the Rondout pressure tunnel have appeared in the following issues of the Engineering Record: Sept. 17, 1910, page 312; Oct. 15, 1910, page 441, and June 17, 1911, page 660.

When the tunnel had been unwatered after the test it was found that the lining throughout was in good condition and unruptured except in a stretch extending for about 1600 ft. north of shaft 4. Within the limits of the Binnewater sandstone and High Falls shale the lining was intact where the steel drip-pan construction, shown in the drawing, had been employed. Elsewhere in this formation a crack of from 1/16 to ⅛ in. had occurred along the mid-diameter on both sides of the tunnel. Opposite the mud seam on the incline, where no reinforcement had been placed, cracks had also developed at the springing line. All along the horizontal mud seam from the top of the incline to the mud seams where reinforcement had been placed longitudinal cracks had developed at the springing line. These cracks increased in width toward the north from 1/16 to 3/16 in., and near the north end there were numerous diagonal shear cracks. The lining in the two vertical mud seams, which had been reinforced by ½-in. steel boiler shells, and in the stretch between them was intact.

Later investigations showed that the arch was thoroughly filled with grout, that the grout was of sufficient density and strength to withstand the pressure and that there were no signs of movement in the grout or between the grout and the rock. It seemed probable that in the limestone there had been a general movement of the rock, due to hidden mud seams, and that in the sandstone the rock had simply been compressed a little in taking up the load.

A thorough examination and consideration of all the conditions involved led to the belief that the integrity of the tunnel was in no way involved by these breaks. There was also every reason to believe that the leakage would not increase with time, but would rather be considerably reduced. The question of repair, therefore, was entirely an economical one—would it pay to spend a certain amount of money to save a certain amount of water? On these grounds it was decided to proceed with the work.

Types of Reinforcement Considered

Various types of cast-iron and steel reinforcement were proposed for the repair work. The limiting features of the problem were (1) the small opening through the shaft, which made it necessary that the reinforcement be taken down in sections and assembled in the tunnel; (2) lack of space in the tunnel for assembling and riveting these sections, and (3) the desire to reduce the diameter of the tunnel as little as possible, which rendered necessary a small clearance between the finished reinforcement and the original lining. The last consideration made it difficult to transport the finished reinforcement through the tunnel and practically required erection in place. Cast-iron was eliminated on account of the weight necessary to resist the high pressure and on account of the excessive reduction of the waterway.

Choice finally narrowed down to two methods of reinforcement. The procedure in the first scheme was to roll 15-in. channels to a radius 2 in. less than the original radius of the tunnel, take these down the shaft in three sections and rivet them into a single ring at some convenient point in the tunnel. The completed rings were then to be erected flange to flange and the flanges riveted together. The annular space of 2 in. outside the rings was to be grouted and a lining of 7½ in. of concrete placed inside the rings. This would result in a reduction of the diameter of the tunnel through the reinforced portion from 14 ft. 6 in. to 12 ft. 9 in. The second plan was to substitute for the channels steel plates with inside flanges, the entire work of erection and riveting to be done in place. The channel reinforcement was finally adopted on account of its greater strength and rigidity and greater ease of erection. The design provides for reinforcement and protective inside and outside lining of concrete and grout.

A proposition was made by the contractor to connect the three 15-in. channel segments into a single ring by means of electric welding instead of riveted butt straps, as had been intended. The substitution of this method was suggested by the Electric Welding Company of New York, which claimed greater efficiency for the welded joint, and that the welded joint could also be made more expeditiously in the confined space available than the riveted joint.

Test welds of full-sized rings at the Watertown Arsenal developed a strength of 46,000 lb. per square inch of cross-section of the original channels. This was considerably in excess of the strength of a riveted butt strap. It was accordingly decided to adopt electric welding for joining the three segments into a single ring.

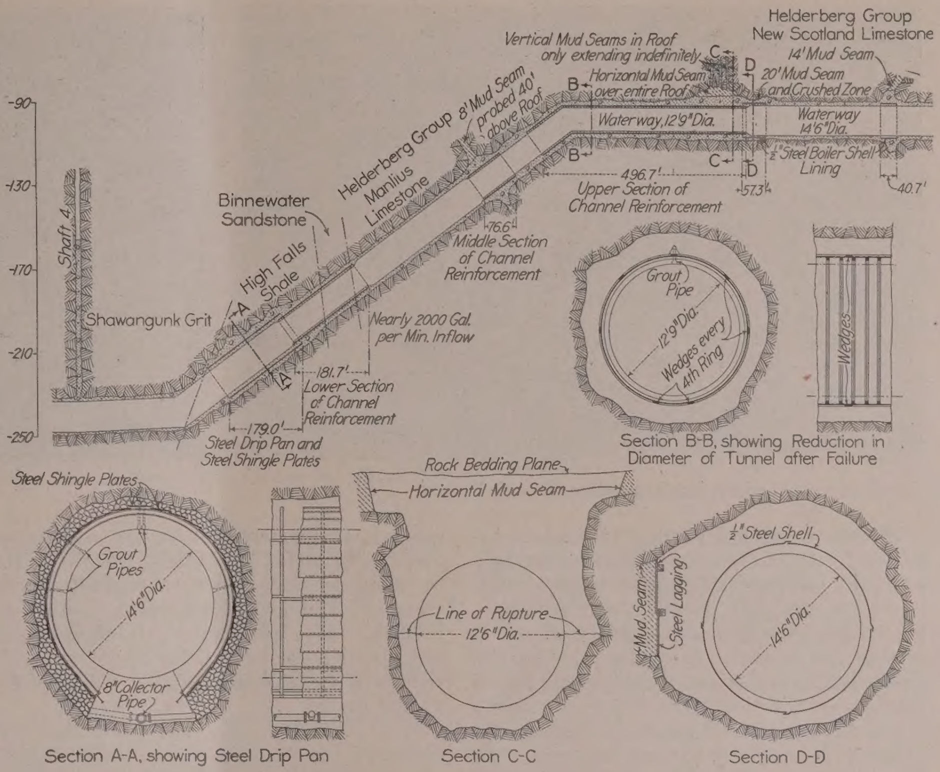
The total length of the tunnel to be reinforced was 758¾ ft. divided into three stretches of 185, 77½, and 496¼ ft. respectively. For this length 606 15-in. 55-lb. channel rings were required.

THREE-SEGMENT RINGS

After rolling and cutting the channel segments to the proper length they were assembled on a template and rigidly fastened together with two steel bands. No offset of more than ⅛ in. was allowed between butting ends. The rivet holes were then drilled through a second template. As the rivet pitch was 4.32 in. there were 120 holes to each flange, or nearly 150,000 holes in all. The accuracy of the shopwork may be gaged from the fact that no gouging or reaming was required in the erecting and riveting, and very little drifting, rings frequently being erected and bolted in fifteen minutes.

The shopwork was done for the T. A. Gillespie Company in the shops of the East Jersey Pipe Company at Paterson, N. J., under the direction of J. A. Nelson, general manager, and H. B. Elwell, shop superintendent, and under the supervision of Ernst Jonson, engineer inspector, Board of Water Supply.

The section of tunnel just north of the break was fitted up as a workshop for the electric welding, the layout of which is shown in one of the drawings. It was proposed to have six welds in progress at one time, which would require about 1000 amperes of current at from 70 to 90 volts. This power was brought into the tunnel at 2200 volts alternating current, and furnished to a 150-hp alternating-current motor, which in turn drove a 100-kw direct-current generator. As six welders were to be employed, it was decided, in order that there might be no delay in moving or setting up rings, to have four welding platforms and templates each containing one ring of three joints, or twelve joints in all. These platforms were so located and spaced that it was possible for segments to be pushed underneath in cars and then hauled up for assembly through spaces between them.



Section B-B, showing Reduction in Diameter of Tunnel after Failure

Section A-A, showing Steel Drip Pan

Section C-C

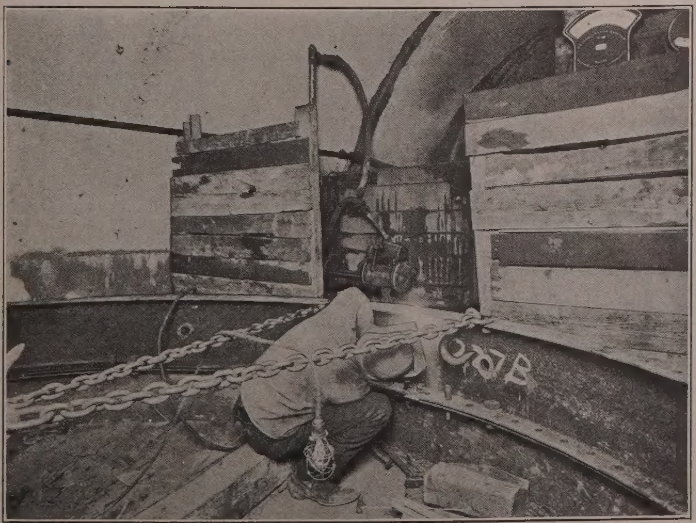
Section D-D

Location of Mud Seams and Type of Reinforcement for Ruptured Tunnel

On each platform a ring was assembled, temporarily fastened together with butt straps and securely braced as a template. As a completed ring is only 4 in. less in diameter than the tunnel it was decided that all finished rings should be moved in a horizontal position on the mid-diameter of the tunnel. By cutting niches in the concrete the platforms were placed at such an elevation that there was room for a template, for a ring assembled on top of it and for moving a finished ring over both. As the finished ring was a "white elephant" until erected, similar niches were cut beyond the welding platform to provide for three storage spaces with a capacity of three rings each, or nine rings in all.

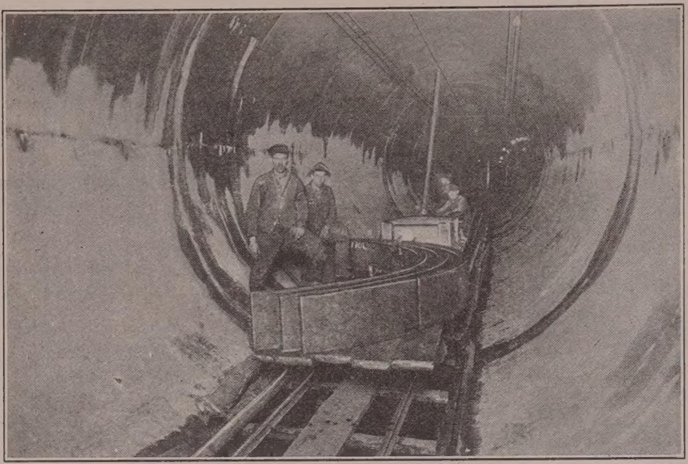
ELECTRIC WELDING

The three segments of a ring were bolted together side by side for convenience in handling and were lowered down the shaft, unbolted, hauled to the welding platform and set over the templates by means of chain hoist and trolley. In setting the segments up over the templates four points were to be observed—(1) that the backs of the adjacent channels at a joint should be flush so as not to bring an eccentric stress and consequent bending movement on the completed weld; (2) that the tops of the flanges should be flush so that the ring would fit tightly against the next one when erected; (3) that the rivet spacing across the joint should be correct; and (4) that there should be an open space of at least 1/16 in. at the joint to permit "burning through" while welding.

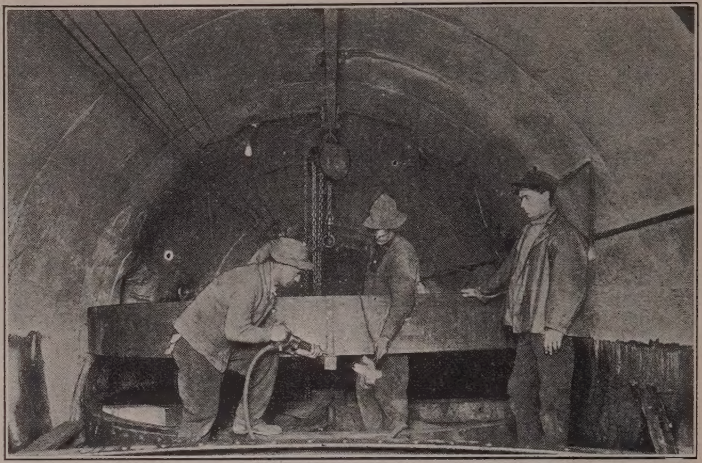


Making an Overhead Electric Weld in Steel Ring

Bull Riveter at Work on Channel Flanges



Carrying Segments through Tunnel



Chipping Off a Welded Ring

Electric welding depends on the heat generated by an electric arc, the negative pole of which is a specially coated metallic electrode and the positive pole the material to be welded. As the electrode is advanced toward the point at which the weld is to be made the current arcs across, setting up an intense heat, which melts the metal in the electrode and fuses it into both sides of the original metal, filling the gap between. The union is thus chemical and not mechanical and the process should technically be known as fusing.

Force welding was, of course, out of the question. Oxyacetylene and carbon arc welding was not practicable because of the overhead work, and also because of the strict requirements for high strength. So it seemed that this work could be best done by means of the metallic electrode arc with a refractory-flux coating on the electrode.

Considerable mechanical skill is necessary on the part of the operator, as the arc fails as soon as the electrode comes in contact with the metal in the weld or with the material being welded, and the sudden cooling causes it to stick, with the result that a lump of partly cooled metal is deposited which is not properly fused. On the other hand, failure to keep the electrode close enough will result in insufficient heat and poor "burning in" of new metal to old and slow work. Due to the intensity of the ultra-violet rays generated by the arc it is impossible to watch it for any length of time with the naked eye. The operators are protected by a "light box" made of thick colored glass and depend a great deal on the senses of hearing and touch.

The direct-current voltage was from 70 to 90 volts, and 150 amperes were used for each weld. Due to the great fluctuations in demand as the electrode approaches the weld and the continual short circuits due to unavoidable contact between them a large resistance is used between the generator and electrode. The adjustment of the resistance, with consequent regulation of the arc, is one of the niceties of the process, for if the arc is too hot the weld will be burned and brittle, and if too little heat be generated the weld will not be properly burned in.

STAGES IN WELDING

The preliminary experiments had shown that the process of welding tended to contract the channels slightly. As the flanges were the weakest part it was considered advisable to weld them first so that they would be put in initial compression by the welding of the web and so be in a better position to resist the tension caused by the bursting pressure when the tunnel was in service. The procedure was therefore as follows: (1) Weld top of top flange; (2) weld bottom flange and inside of web; (3) weld outside of web; and (4) weld bottom of top flange.

Welding began Dec. 28, 1912, and was continued daily with a few interruptions due to lack of material until April 4, 1913, when it was completed. Two shifts of five men each were employed, and the average progress was five rings, or fifteen welds per shift. The work comprised 1824 welds, making a total length of more than 6000 ft. of welding. The speed of welding by this process varies with the thickness of the plate, with the direction in which the strings of metal are run, and with the angle at which the weld is made. In this case 84 per cent of the weld was vertical, 8 per cent overhead and 8 per cent downward. The welding was done by the Electric Welding Company, of New York City, for whom Wilhelm Schenstrom was engineer in charge of the work.

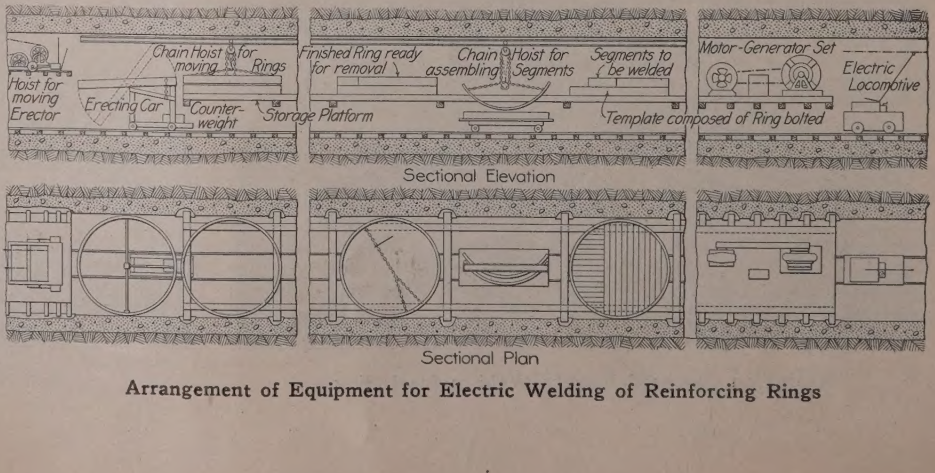
ERECTION OF RINGS

The transportation and erection of the ring were the critical parts of the work, as the ring, once welded, blocked all further work until set in place. It was impracticable to find storage for more than ten completed rings, and on account of its small clearance the ring in transit or erection required the temporary removal of all power lines. It was thus necessary to remove the rings as fast as welded and erect them immediately so that the power lines could be restored and the riveting and calking allowed to proceed. This was accomplished by means of the erecting car shown in the drawing.

The erecting car was framed of heavy timber. It carried a 3-in. solid steel shaft approximately on the axis of the tunnel and parallel to it. At the end of this shaft was fixed at right angles another shaft of a length equal to the diameter of the ring, with clamps on each end for holding the ring. By means of the first shaft the ring could be rotated and by means of the second it could be tilted through 90 deg., or from a horizontal to a vertical position. The finished rings were carried from the storage platform to the erecting car by means of the chain hoist and trolley, and were clamped tight in a horizontal position. The car was moved along the level and lowered down the incline by means of a hoisting engine. The ring was then tilted into a vertical position, rotated as far as necessary and bolted to the steel already erected. Any up and down or side movement required for erection was obtained by moving the track.

Although each ring weighed 1¼ tons, one man could swing it into position readily. In this way rings were frequently loaded, carried through the tunnel for more than 1000 ft. and erected inside of half an hour. Three men were employed loading and five erecting and bolting. No difficulty was experienced in erecting ten rings, or 12½ tons of steel, regularly in an eight-hour shift, and it is a tribute to the care with which this part of the plant was planned and designed that there was never any delay on account of erection.

After bolting, every fourth ring was wedged against the concrete with steel wedges at the mid-diameter and at three points in the upper half and two in the lower, making seven in all. This was not only to hold the steel in position during erection, but also to resist the flotation during the grouting. The alignment of the steel was governed by these wedges.



Arrangement of Equipment for Electric Welding of Reinforcing Rings

Erecting was started on Dec. 29, 1912, and completed on April 5, 1913, a total of about 800 tons of steel being placed.

RIVETING

Ten rings were erected daily, requiring 120 rivets each, or 1200 rivets per day. Fortunately the total amount of rivets, 73,000, was enough to warrant the installation of special appliances. A Hanna portable "bull" riveter, operated by compressed air, was installed mounted on a special carriage, as shown in one of the photographs. This machine had a pressure at the jaws of about 80 tons, which was very useful in drawing up flanges which were not in contact. The rivets were heated in a Hoskins electric muffle furnace, which had a capacity of about 600 rivets per eight-hour shift. The furnace was supplied with current at from 12 to 55 volts and gave satisfaction, although the firebox required frequent replacement. The use of coke heaters would have been almost impossible on account of the excessive fouling of the air. The driving of a complete ring of 120 rivets required about seventy-five minutes on the average, and about half an hour was consumed in each move. Riveting was started Jan. 9, 1913, and completed April 17, 1913. Of the 73,000 rivets driven not more than 100 were defective, and of these the greater part were defective heads due to imperfect or careless adjustment of the extension arm.

CALKING

In order to make the reinforcement tight against heavy pressures the flanges were all calked as in a riveted steel pipe. Due to the thickness of the flanges and the absence of a calking edge, this work was very heavy. Air-operated pneumatic hammers were used, and for the difficult overhead calking a heavy bit was set in a stope drill from which the rotating mechanism had been removed. A 10 x 10-in. timber was placed on a platform along the axis of the tunnel and on this moved a specially designed rest which allowed the stope drill to be rotated around the top third of the circumference of the ring.

Calking was started March 27 and carried on by two shifts of six men each until its completion on May 8. A total of about 24,000 lin. ft. of calking was done.

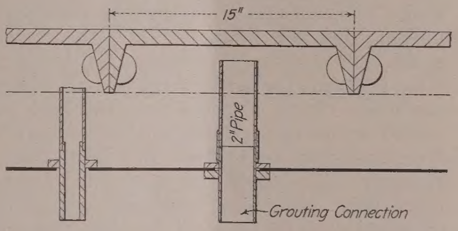
There were two kinds of grouting to be done—first, the 2-in. annular space between the reinforcement and the original lining, and, second, the fissures in the lining and in the rock, through which the ground water reached the tunnel and through which water might be expected to escape later if it should get by the reinforcement. It was advisable to separate these steps, as the former could be better done with very low pressures of from 20 to 30 lb. to the square inch, and the latter required pressures as great as 300 lb. per square inch.

Infiltration into the tunnel, which was about 400 gal. per minute prior to grouting, was reduced to 2 gal. per minute.

CONCRETE LINING

The remainder of the work consisted in placing a 7½-in. concrete lining inside the steel for the double purpose of protecting it from corrosion and of providing a smooth waterway. The lining was placed in the usual manner for pressure tunnels, which has been described in former articles in the Engineering Record.

After the completion of the repairs the infiltration into the tunnel was about 34 gal. per minute, compared to 56 gal. per minute before the former hydrostatic test and 400 to 500 gal. per minute during the progress of the repair work after the break. The tunnel was subjected to a second hydrostatic test between Oct. 20 and Nov. 11, 1913, and the total leakage under service conditions was found to be about 850 gal. per minute, as compared with an estimated leakage of 3500 gal. per minute prior to the repair work. This is considered highly satisfactory, as the first leakage on the Wallkill siphon, a Catskill Aqueduct pressure tunnel of about the same length but of less depth, through tight homogeneous rock, was about 1100 gal. per minute. A much smaller leakage may be expected when the tunnel has been in service some time and the ground water is restored, as the leakage on the Wallkill siphon under a long-time test decreased to less than 400 gal. per minute.



Relation of Rings to Grout Pipes

The original construction and repair work were both done by the T. A. Gillespie Company, of New York. R. J. Gillespie was general superintendent, Francis Donaldson, chief engineer, and Luke White, superintendent. The repairs were made under the supervision of the Board of Water Supply of the City of New York, of which J. Waldo Smith is chief engineer; R. N. Wheeler, department engineer; the author, division engineer, and Milton H. Freeman, section engineer.

Florida's Humble Topography

THE highest accurately determined point in the whole State of Florida, Mount Pleasant, in Gadsden County, is 301 ft. above sea level. This bashful peak is in the center of a deeply dissected upland running across the northern part of the State, crossing Gadsden, Walton, Santa Rosa and Escambia Counties, according to a press bulletin issued by the U. S. Geological Survey. Thirty miles southeast stands Tallahassee, the capital, 205 ft. above sea level, on the remnant of an upland which has been isolated by erosion. The highland in the center of the peninsula forming the southern part of the State boasts a few elevations greater than 200 ft., but the area at the extreme south, 150 miles long by more than 100 miles in average width, lies, in general, less than 50 ft. above sea datum.

EXTREMELY HIGH WAVES recently broke heavy plate glass in the lantern—which is 130 ft. above high tide level—of the Tillamook Rock light station, off the coast of Oregon, according to an official report from the lighthouse keeper, who recorded the highest waves that had occurred in the twelve years of his service there. The gale had raged for more than fourteen hours and a maximum wind velocity of 84 miles per hour was indicated.

Recommendations for Concrete-Road Building

Principles Advocated by the National Conference on Concrete-Road Building

FUNDAMENTAL principles of concrete-road construction were adopted at the National Conference on Concrete-Road Building, which was held in Chicago Feb. 12 to 14. As noted briefly in the Current News section of this journal last week, the salient features of the recommended practice are:

1. The aggregate should be clean and hard.
2. The sand should be coarse and well graded.
3. A rich mixture should be used.
4. The materials should be correctly proportioned.
5. The materials should be thoroughly mixed.
6. The inspection should be intelligent and thorough.
7. When in doubt, reinforce the pavement.
8. The subgrade should be of uniform density, thoroughly compacted and drenched with water immediately before placing concrete.
9. The concrete should be of a viscous, plastic consistency.
10. After placing, the concrete should be immediately covered and kept moist and not opened to traffic for four weeks.

ADDITIONAL SUGGESTIONS

Many of the additional recommendations agree with prevailing practice or have been sufficiently dealt with in the committee reports of the conference, abstracts of which will be found in the Engineering Record of Feb. 21 and in this issue. However, some points deserve emphasizing.

For all roadways less than 20 ft. in width is recommended a cross-section showing a slab thicker in the middle than on the edge, built upon a flat subgrade. The dished subgrade recommended in the report of the committee on contraction and expansion is not mentioned.

Minimum widths of 10 ft. for single-track and 18 ft. for double-track roads are approved. In the country a crown of 1/100 of the width is recommended; for city pavements the specified crown is 1/70.

The use of suitable reinforcing at the junction of shoulders and metal, as stiff gravel or stone chips, is urged.

Joints should be avoided, if possible. Reinforcing the slab is the alternative suggested. In fact, it is urged that all roads exceeding 20 ft. in width be reinforced with wire fabric whose co-ordinate cross-sectional areas are 0.038 sq. in. and 0.049 sq. in. per foot, the greater section to be applied transversely. The type of filler recommended, if joints are necessary, is that made of two 3 x 3/16-in. strips of metal separated by about ¼ in. of prepared felt.

Adding materials or water to mortar or concrete that has partly hardened and carrying on work during freezing weather are practices strongly condemned. If the temperature should drop below 35 deg. Fahr., the water and aggregates should be heated and the work protected from freezing for at least ten days. Concrete should not be deposited upon subgrade that is frozen.

The conference recommended the standard specifications for pavements and roadways of the American Concrete Institute.

Arrangement of Concrete Products Plants

Attention Called to Correct and Incorrect Principles of Design and Operation

COMMENTING on the healthy growth of the concrete products industry in an address before the American Concrete Institute in Chicago, Feb. 18, E. S. Hanson, editor of the "Cement Era," called attention to the good and bad features of the layout of existing plants and suggested the standards that should govern in rational design. An abstract of these recommendations follows.

Every concrete plant—whether supplying block, tile, post, or structural or ornamental member—should be built with a view to permanence, service and a certain measure of sightliness. The kilns should be of concrete to insure permanence. If the plant is manufacturing blocks, the kilns can easily be constructed one at a time after the main part of the plant is up and the machines are in place, using blocks for the walls and a simple form of reinforced-concrete arch for the roof. This roof can be built up over a rough wood form, or can be constructed without centering on some one of the patented types of metal designed for this purpose. In the latter case, however, it should be carefully back-plastered in order to preserve the metal from the corroding effect of the steam. The curing room of building paper will ultimately be crowded out entirely by the demand for permanent construction.

Curing chambers are separate units and can be added on from time to time as needed, provided only that the plant is so designed beforehand as to make this possible. A street on one side and a railroad siding on the other might prove an effectual bar to the construction of more curing space without an entire reconstruction of the plant, and this contingency should be avoided.

Design of Doors

It is quite important that the doors of kilns be so constructed that they may not be in the way when open. An outward swinging door is the worst imaginable, though it is still to be found in a few plants. It not only is likely to become a frequent obstruction, but often is so loosely made that it sags on its hinges and will not close the kilns tightly. A sliding door has either to cover adjoining kilns or allow of opening only one-half the chamber which is in service. A door which is removable and held in place by a cleat resting in sockets gives good service, but is unnecessarily clumsy to handle.

Of the various types of rigid doors, the upward swinging door is perhaps the best for the inner end of the kilns. Such a door can be held up by a simple latch, is entirely out of the way, is easy to handle and its weight keeps it closed when down. At the outer end of the kiln an upward-sliding door can be used to advantage. In some cases this is counterbalanced with a weight, making it very easy to operate.

Canvas curtains can be made practically steamtight by having stationary strips nailed to the walls at the sides of the curtains and other strips hinged to these with screen-door hinges. When the curing room is filled, the curtain is lowered and these movable hinged strips are turned down over the edges of the curtain, the spring hinges holding them tightly in place and making a joint practically steamtight. In addition to this precaution, the canvas curtains may be painted so that no steam can penetrate them.

A concrete floor throughout the plant will be found well worth while. In addition to its durability, its rigidity and smooth surface commend it for the efficient moving of product, while it serves well in many cases as a molding floor, or as a curing floor for setting out sewer pipe or large tile.

Having the mixer elevated above the working floor is a decided advantage in that the delivery of the concrete can be done mostly by gravity. It involves, however, a larger expenditure of power for elevating the materials, and in many cases somewhat increases the labor cost because of the fact that a man will either have to run back and forth very frequently or else spend more of his time than is necessary in watching the mixer. While the mixer could very well be set some little distance from the working floor, there are not many instances where it could to advantage be placed on an upper floor, entirely separate from the main workroom. In a large plant, however, this might be advisable, where more than one mixer was required to be in operation practically all the time, necessitating the continual attention of a man.

Slush Process

Mr. Hanson described, among others, a plant devoted to the manufacture of concrete blocks by the slush process. By this process multiple molds are carried on cars and the facing plates carried on the bottom of each mold. The concrete is poured so wet that steam is not generally used for curing. The curing chambers, therefore, are not steam chambers in the ordinary sense of the word, such steam as may fill them being vaporization of excess water from the blocks themselves. Closed steam pipes for heating run under the tracks in the curing chambers, generally being used only in seasons of low temperature. At other times the cars are simply run into the chambers and closed to keep off the sun and wind, so that the water may not evaporate too rapidly. The cars are run onto the transfer track under the mixer and filled by means of a chute. In front of each curing chamber is a length of track sufficient for a car. This is for the purpose of leaving the car of blocks outside the curing chamber for an hour or two, until the initial set has taken place and part of the water has evaporated.

Automatic Handling of Materials

In another plant the automatic handling of materials has been given particular consideration in the construction and equipment. This has been accomplished very largely by the use of screw conveyors. The sand is hauled in by teams and is dumped onto a grating, through which it falls to a hopper situated below the ground level. This hopper delivers to the trough of the screw conveyor, which runs under the floor of the main workroom, and from which the material is elevated by a bucket conveyor to the second floor of the building. The bucket conveyor delivers directly to the screen above a hopper, which in turn delivers to another screw conveyor, at the discharge end of which is a measuring box by which the material is measured before going into the mixer. This box has a section of glass in the front, so that the mixer operator can see the material fill up to a height predetermined for the particular mix he is using. In order to allow him to do this, the entire machinery which handles the gravel is placed at his instant control by electric-motor drive.

The entire conveying machinery is operated as a unit, so that when one part is stopped, all of it stops, obviating the possibility of the material piling up at any one point. It is so arranged, however, that that part of it which takes material from the outside and elevates it to the second floor can be run independently, if desirable, without running that part which feeds the mixer. By this arrangement a considerable supply of gravel can be stored on the second floor if it is thought necessary to provide against temporary closing of the sand pit or for any other reason.

A screw conveyor is again used to convey concrete from the mixer to the tile machine. The mixer is of the tilting type and delivers directly into the trough of the conveyor, the latter in turn delivering directly to the boot of the tile machine.

An Efficient Layout

Radical features have been introduced in a third plant. For instance, the kilns do not open into the main workroom, the object being to keep the steam from the curing rooms and entirely away from the workroom. For this reason the kilns are made parallel with the workroom instead of at right angles with it. It is also designed to approach a square as nearly as possible, as this is usually the most inexpensive shape of building to construct. A number of windows in the shed which covers the transfer track not only admit light for the workmen, but also open readily for the egress of steam. This plan was drawn with the intention of having it located close to the sand pits, with an inclined track from the pit to the top of the building and dumping directly into the screens. The sand to be used is dropped directly into the bin, while the large stone are carried away in a chute and emptied outside the building. Both the sand and cement storage can be made with partial hopper bottoms and connect directly with the mixer. Provision is made for the delivery of coal under the cement storage, so that both materials can be delivered from cars on the railroad track. The sand can also be delivered in the same manner should the sand pit be exhausted, or if for any reason it is desired to use a different supply.

Denver Water Situation Unchanged

ONCE more an attempt to settle the Denver waterworks controversy has been defeated by vote of the taxpayers, and the situation is entirely unchanged. Interviews with prominent citizens as printed in the "Rocky Mountain News" indicate the people voted under a misapprehension, suspecting the retail-association plan involved something other than an honest effort to get municipal ownership by appraisal. It appears the consensus of opinion is that the existing plant will be purchased rather than duplicated, a procedure the Public Utility Commission holds it is bound to pursue if it cannot purchase the plant for $7,000,000.

Senator Patterson proposes to offer Colonel Goethals $25,000 a year to build new works, or at least to get him to recommend an engineer in the event a "fair" price is not accepted by the company.

Steel Highway Bridge Failure

Tractor Weighing 18 Tons Causes Deflection of Bridge Stringer and Distorts Structure

By N. B. GARVER
Associate in Civil Engineering, University of Illinois

AN old steel highway bridge over Hayes drainage ditch in Tuscola Township, Douglas County, Illinois, failed recently under the load of an 18-ton motor tractor. The heavy load from the rear wheels of the tractor deflected the bridge stringers and distorted the structure. The writer witnessed the failure and, as the safety of old steel highway bridges under present-day traffic is a subject of considerable importance since the advent of the modern tractor and the motor truck, the following account of the failure is given:

The tractor was of the 22-hp double-cylinder Avery type. According to information received from the manufacturers, the tractor weighed about 18 tons with the boiler, tanks and fuel bunkers filled. The wheel spacing was about 13 ft. center to center of axles, 7 ft. center to center of rear wheels and 5 ft. center to center of front wheels. In the Engineering Record of May 6, 1911, page 515, there is a letter to the Editor by Paul Funderhide in which it is stated that an Avery engine of this type has about four-fifths of the weight concentrated on the rear axle.

DESIGN OF BRIDGE

The bridge was of the so-called leg or bedstead type, and it was built about 1895. A sketch of the truss, a floor plan, and some other details are shown in the accompanying drawing. The township commissioners stated that three of these bridges were purchased at one time for the sum of $1000.

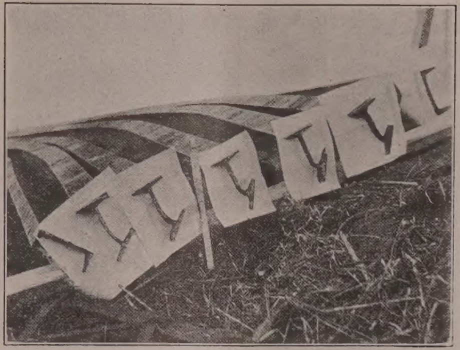
Its floor consisted of 2½-in. transverse oak planking, not fastened to the stringers but held in line by a 2 x 2 x ¼-in. angle, spiked to the planks at each side of the roadway. The stringers were fastened to the floorbeams by ½-in. round-headed, machined bolts, two at each end of the I-beams and one at each end of the channels. The channels had holes punched in the top flange about midway between the ends of their spans.

The engine was not moving under its own power, since it had partly slipped down the approach grade the previous evening and the fire under the boiler had been allowed to go out. The power to move the engine forward was obtained by winding a rope around the flywheel and tying the end of it to the rear axle of a wagon to which a team of horses was hitched. By this means the engine was moved forward by short stages.

LOADING OF BRIDGE

The engine came upon the bridge in about the position shown in the drawing. It will be noted that one of the heavy wheels came upon the floor near one of the channel stringers. The channel deflected under the load and the planks lifted from the stringers at the other side of the roadway. When the rear wheels were about 2 ft. from the abutment the channel had deflected about 2 in. and the planking had lifted about 12 in. at the other end.

When the engine had reached this position it stopped and the stringers gave way gradually—the channel under the heavy wheel first, followed in succession by the intermediate I-beams and the channel on the far side of the roadway. The final position of the stringers was about as shown by the dotted lines in the drawing. The engine stopped with its front wheels in the middle of the second panel, the rear wheels about 5 ft. from the abutment, and the back platform resting on the abutment.



Bridge Stringers after Failure

DAMAGE

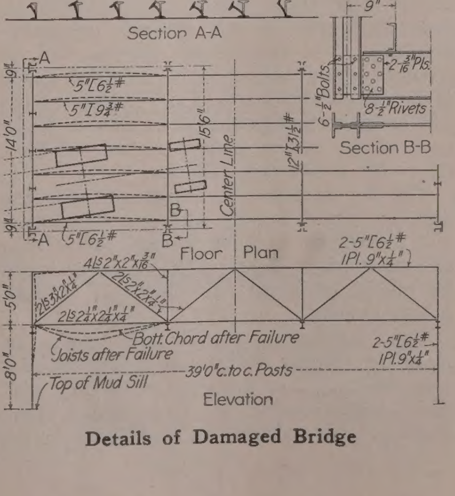
The damage to the truss nearest to the engine is indicated by the dotted lines in the drawing. The bend in the bottom chord was due to the fact that the floor planks projected over it. The end post was pulled in by the stringers and bottom chord. The bend in the top chord was due to the bending of the end post, as the connection was rigid. The other truss was uninjured.

At the ends of the stringers the ½-in. bolts held in every instance, the result of which may be noted in the bent webs and flanges shown in the photograph. The stringers twisted through an angle of nearly 90 deg.; the I-beams showed a very regular bend, while the channels had a sharp bend at the points where holes were punched in the flanges. The point of maximum bending was about 5½ ft. from the abutment.

UNIT STRESSES

If the usual assumption is made that the concentrated wheel load is distributed to two stringers by the plank flooring, the extreme fiber stress in the I-beam nearest the channel is 30,000 lb. per square inch if the wheel is placed 2 ft. from the support (about the position when the floor failed). The extreme fiber stress in the channel under the same assumption is 49,000 lb. per square inch.

Observations of the failure of this bridge lead to the following conclusions: Plank floors should be fastened to the stringers at the ends in order to distribute the concentrated loads more widely. A plank floor 2½ in. thick will not distribute a concentrated load equally between two stringers. Holes punched in the flange of a beam or channel materially weaken its strength, and when they are necessary due allowance should be made in the design. I-beams should be used for all stringers.



Details of Damaged Bridge

Production of Pig Iron

THE production of pig iron during 1913 in the United States is given in detail in a special bulletin recently issued by the bureau of statistics of the American Iron and Steel Institute. The total production in the United States was 30,966,301 gross tons, as compared with 29,726,937 in 1912, and 27,303,567 in 1910. For the last half of the year, however, the production was 2,010,903 less than for the first half and 1,176,964 less than for the last half of 1912. The outputs of the first six States (New York and New Jersey being classed as one) show no material change and continue to constitute 94 per cent of the entire output. Pennsylvania continues to lead, with 12,954,940 gross tons, or 41.84 per cent of the total.

The production of bessemer and low-phosphorus pig iron was 11,593,385 tons, a decrease of 70,630 tons from 1912. The production of bessemer pig iron alone was 11,276,567 tons in 1913 and 11,381,656 tons in 1912. Of basic pig iron, including a small tonnage of charcoal pig iron of basic quality, the output was the greatest in our history, 12,537,746 tons, against 11,417,886 in 1912. The production was less for the last half of the year than for the first half.

During the year five blast furnaces were built and ten abandoned or dismantled. The five have a greater capacity than the ten had. On Dec. 31, 1913, three furnaces were in course of erection and eight were being rebuilt.

Grouting Water-Bearing Fissures in Rock

IN constructing the deep pressure tunnels of the Catskill Aqueduct for the water supply of New York City many rock fissures had to be grouted. The most difficult piece of work of this kind, according to a paper in the "Transactions" of the American Institute of Mining Engineers by Francis Donaldson, was at the Hudson River siphon, 1100 ft. below tidewater. About 150 ft. from the east shaft the heading cut a water-bearing seam, discharging about 300 gal. per minute. The full flow did not develop until the cut was blasted. It then became necessary to grout against a hydrostatic pressure of 500 lb. per square inch with no solid rock to which the grout pipes could be connected. This problem was finally solved by building a concrete bulkhead 8 ft. thick across the heading. The concrete for this bulkhead was of a 1:2:4 mixture, and was heavily reinforced with rails set into holes drilled laterally in the sides, roof and floor of the tunnel. Grout pipes leading into the fissure were set through this bulkhead. After the concrete had set for a week grout was forced into the fissure first by the pneumatic process with a high-pressure air compressor and finally with a high-pressure plunger pump forcing water instead of air into the grout tank. Pressures were reached up to 1000 lb. per square inch.

Contraction and Expansion of Concrete Roads

Topics of Moisture Expansion, Design of Sub-Base, Reinforcement and Joints Covered in Committee Report at National Conference on Concrete-Road Building

THAT change in moisture content influences the behavior of concrete-road slabs more than change in temperature, under normal exposure, that a concave sub-base is preferable for all conditions, and that it is wise to reinforce the slabs at decided changes in grades and when the width is greater than 25 ft., are the principal conclusions of an unusually complete report presented at the National Conference on Concrete-Road Building in Chicago Feb. 13. The authors of the report, constituting the committee on contraction and expansion of concrete roads, are R. J. Wig, of the U. S. Bureau of Standards; N. H. Tunnicliff, of Davenport, Ia., and W. A. McIntyre, engineer of the Association of American Portland Cement Manufacturers. An abstract follows.

INFLUENCE OF MOISTURE

Though the possibility that concrete road slabs might exhibit a phenomenon often noted in the laboratory—swelling in water—has been suggested from time to time (see Engineering Record, July 19, 1913, page 76), the real fact is just gaining recognition. Measurements made by the Bureau of Standards on a concrete road on Morris Turnpike, near New Village, N. J., covering the period from October, 1912, to October, 1913, and in Nazareth, Pa., from June to October, 1913, show that the linear change in the concrete is not in accord with the temperature change. One slab was placed June 9 and measurements were started June 18. Between the latter date and Aug. 19 there is a considerable shortening of the slab, although the temperature had increased during this period.

The majority of a number of other tests gave like results, which accorded approximately with those noted by many observers, of which the University of Michigan may be taken as typical. There it has been observed that neat cement hardening under water will, at the end of three years, show an .11 per cent increase in length and, if then exposed in air for two months, will contract .16 per cent, attaining stable volume when .05 per cent smaller than its original size. The corresponding contraction for neat cement hardening in air is .322 per cent. If alternately exposed to air and water, the cement will assume volumes indicated by a sawtooth curve. Mortars (1:3) and bars exhibit a similar phenomenon, though to a less degree.

In experiments conducted by the Office of Public Roads it was observed that the rate of shrinkage of the moistened specimens, when allowed to dry out, approximated that of the air-cured specimens, so that one of the effects of moistening the concrete for the first few days was to delay the shrinkage. A specimen, which was kept in a moistened condition continuously, although not expanding progressively, showed expansion up to the last measurement.

The results of experiments made by the Bureau of Standards with neat cement prisms 1 x 1 x 13 in. are given in the table.

The condition and character of the sub-base affect the support of the road, the amount of moisture transmitted to and from the concrete, the action of frost, and the friction on the slab bottom—all of which factors influence expansion and contraction. If the slab of concrete is not uniformly supported, sections may be stressed, causing tension or compression in an amount sufficient to exceed the strength of the concrete, resulting in cracking. The amount of moisture transmitted to and from the concrete through the sub-base depends upon the porosity and density of the material in the sub-base. The action of frost can only occur in a sub-base containing moisture in localities in which the temperature gets sufficiently low to freeze to a depth of the sub-base. The effect of freezing and thawing is to place the slab in stress which may cause cracking. Observations indicate that the cracks usually occur during a thaw rather than during a frost, and this may be explained by the fact that the heat may be transmitted more rapidly through the shoulders than through the center of the road, causing unequal settlement.

The effect of friction on the bottom of the slab is to restrain movement in the concrete; therefore the tendency to volumetric change due to change in moisture content and thermal change would be restrained. This restraint would set up stresses in the concrete probably proportional to the restraint. The report cites the performance of a slab which moved only half the theoretical expansion, probably because of this friction. The longitudinal grade of the road would affect the friction on the bottom of the slab. On a 5.2-per cent grade on Morris Turnpike a slab 30 ft. long had moved downhill, the joint at the upper end of the slag being decidedly open, while the lower end of the slab slid upward on the adjoining slab.

LINEAR EXPANSION OF NEAT CEMENT PRISMS IN WATER AND IN AIR

Number of cements	Manner of storing	Age of test piece	Change in length per unit of original length
6	In water	30 days	+ 0.00095
8	In water	30 to 60 days	+ 0.00105
4	In water	120 days	+ 0.00113
20	In water	6 to 9 months	+ 0.00152
6	In air	30 days	— 0.00150
15	In air	30 to 60 days	— 0.00167
4	In air	120 days	— 0.00211
40	In air	6 to 9 months	— 0.00285

CONCAVE SUB-BASES

In discussing the application of present knowledge to the prevention of cracks the committee compiled a series of questions which it undertook to answer. Here the advice is given to "dish" all sub-bases, in order to throw the slab entirely into compression. The committee believes that to crowned sub-grades may be ascribed many of the longitudinal cracks in concrete roads. The cross-section having the greatest thickness at the center with a dished sub-base offers the greatest resistance.

The two qualities to be reconciled in the mix are great density and low absorption and a strength of 1500 lb. per square inch at twenty-eight days. The concrete should be placed on a wet sub-base and intelligently cured.

The maximum permissible length of slabs, of similar design, on a firm foundation, on a light or flat grade will vary with the climatic conditions. In localities where there is little rainfall, as in Arizona or the San Joaquin Valley in California, the slab length probably should not exceed 25 ft. This length has been determined by an analysis of a report by A. B. Fletcher, State Highway Engineer of California, on the cracking of a monolithic road, together with a knowledge of the results of measurements obtained by the Bureau of Standards.

In localities where the rainfall is intermittent and the ground water rises in cool seasons and lowers in warm seasons, similar to conditions which obtain east of the Mississippi River, the slab may vary in length from 30 ft. to a continuous slab. While the 30-ft. slab may be regarded as a minimum length from economical considerations, the results of experiments indicate that cracks are quite likely to occur under certain conditions even in this length. In many cases it will be observed that even in short slabs only slight motion has occurred at the ends, although there has been considerable motion within the slab, which has been absorbed at points of restraint. A table showing results of observations on the roads of Wayne County, Michigan, which was used in the report, was published in the Engineering Record for Oct. 11, 1913, page 409.

Experiments indicate that the length of slab should vary with the quality of the concrete, the condition of subsoil and subgrade, the bituminous surfacing and the thickness of the slab, all in a manner suggested by the main conclusions of the committee.

REINFORCING TO PREVENT CRACKS

Reinforcement will be of value when the foundations are uncertain, in regions where there is little rainfall and long or wide slabs are desired, or where there is insufficient water for curing, when it is necessary to have a thin slab, when a decided change in grade occurs, when the movement of the concrete is restrained at street intersections or on curves, if joints cannot be placed properly, when the width of the slab is greater than 25 ft., and when it is necessary to crown the subgrade and the slab cannot be made thicker at the crown than at the sides.

If uncertain subsoil conditions tend to cause a settlement on the sides of the road, which is the most common condition, the reinforcement should be near the top. If settlement is apt to occur within the center of the slab the reinforcement should be placed near the bottom, and where the whole of the sub-base is uncertain the reinforcement should be placed near both top and bottom.

Reinforcement to resist stresses due to temperature and moisture changes should be placed near the bottom of the slab. The stresses set up by a change in temperature are insignificant, except in arid regions, compared with those set up by moisture changes, and ordinarily reinforcement to provide for temperature stresses is unnecessary.

EFFECT OF JOINTS IN CRACKING

The committee has no evidence of the failure of slabs in concrete roads due to buckling, crushing or spalling and therefore considers the joint as primarily for the purpose of taking care of contraction. If a concrete road is constructed with the idea that nature will provide cracks, it will crack, but before doing so it will cause a weakening of the concrete for some distance on each side, which may result in a partial disintegration of the road. This is evidenced by short forked cracks appearing running off from the main crack. The cracks thus formed will gradually lengthen with time and will not only be irregular in alignment but they will not be vertical.

The width of the joint controls the longitudinal movement of the slab. A greater width will allow of greater movement, consequently the slab will be more liable to crack upon contracting. A joint should be constructed as a contraction joint with little or no room provided for expansion, as all stresses in the concrete caused by expansion are compressive and may be absorbed by the concrete; later, upon contraction there will be little movement and therefore only small tensile stresses will be induced.

In collating the report the committee found a lack of reliable information necessary to a complete analysis of the subject and therefore suggested the determination of the following factors:

Friction of a concrete slab on different materials and under different conditions of the sub-base; variation in the coefficient of expansion of concrete when under stress; expansion of green concrete due to the chemical action of the cement during setting; effect of change of moisture content in concrete when under stress; bond between steel reinforcement and concrete when the concrete expands or contracts due to change in moisture content; bond between steel reinforcement and concrete when the concrete expands or contracts due to change in temperature; absorption of heat by concrete uncovered and covered with a bituminous carpet; effect of continual alternate expansion and contraction of unrestrained and restrained concrete, and permanent set resulting in wet and dry concrete under various tensile and compressive stresses, and a more accurate determination of the modulus of elasticity of concrete and the coefficients of expansion for different mixtures.

Patches in Concrete Pavements

PATCHES in concrete pavements at Fond du Lac are covered with a ½ in. layer of asphalt as soon as laid to protect the surface until it gets its final set, according to C. M. Powell in discussing a paper at the recent meeting of the Illinois Society of Engineers and Surveyors. The asphalt will not stay permanently, but will protect the surface for two to three weeks, and as a result one can hardly detect the patches except by the variation in color.

Completing Box Section of Sewer under One of the Subways

Utica Passenger-Station Subways

Main Concourse Provided with Seats, Light, Heat and Special Drainage; Separate Tunnel with Elevators for Baggage

IN an article on page 596 of the Engineering Record for May 31, 1913, on the Utica improvement of the New York Central & Hudson River Railroad, the general features of the new passenger station under construction were outlined. Six island platforms are to serve twelve tracks, and two additional tracks are to pass through the middle of the layout, not adjacent to any platform, for fast-freight service. The walls of the station proper, which will be a commodious and ornamental one, are up, but the building will probably not be opened for service for several months.

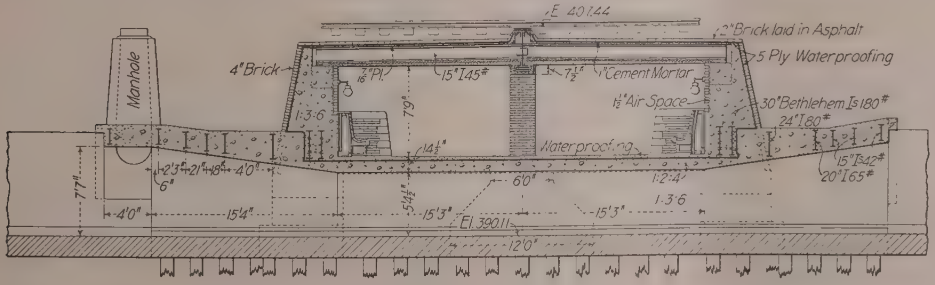
In the design of recent large stations this company has given considerable study not only to the buildings themselves and the track and platform layouts but also to the connections between the two. The narrow, poorly drained tunnel of earlier years is being supplanted in recent designs by broad, well-lighted and well-drained subways, which are furnished with seats and serve as auxiliary waiting rooms, and at the same time provide ample passageway without conflict for incoming and outgoing passengers. The earliest of these developments on the New York Central, that at Troy, was described in the issue of Dec. 20, 1913, page 691. Effort is also made to provide separate runways for baggage and other trucks to keep them out of the way of passengers.

Such an arrangement is being installed at Utica. The passenger subway is 30 ft. wide, with 8-ft. headroom. It extends from the northeast corner of the station, the station being on the south side of the tracks. The subway turns to the west between the station and the first track, entrance from the waiting room being near the corner of the building, and as the main business street is to the west of the station, the subway is carried around the building and up a flight of stairs on the west side. In this way incoming passengers are led around on the right-hand side to the street without crossing the current in the opposite direction. A long, narrow baggage-and-express building is being built east of the station, and about 300 ft. east of the passenger subway a 16-ft. baggage subway with an elevator to each platform (except the inner one) is under construction.

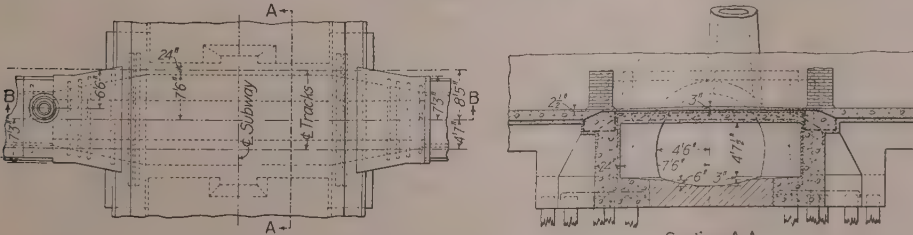
The subways have mass-concrete side walls faced with enameled brick. The floors are of reinforced concrete. Under the tracks the roofs are of I-beam and plate construction, while at the platforms they are of reinforced concrete. All waterproofing is protected with a layer of brick. It was intended to have vault lights over the passenger subway at each platform, and some of them were put in, but it has been decided that artificial lights will be required in any event, and for the remaining platforms the vault lights are being omitted.

DRAINAGE COMPLICATIONS

Complications in the design of the subways were caused by a large sewer running down through the middle of the track layout, about parallel to the tracks. This sewer was part of the original project of relocating the Mohawk River and filling the old channel to make room for the new station. As tentatively laid out at that time, however, a hump in the track layout would have carried the subways up over the sewer, but changes in the plans to meet altered conditions lowered the tracks and subways,



Section B-B



Plan

Section A-A

Details of Crossing of Passenger Subway and Large Sewer so that the subway floors are now considerably below the original top of the sewer.

Drawings are shown of the way this detail was taken care of. At the passenger subway a 61-ft. section of the sewer was removed and replaced with a wide flat box under the subway, warped in at each end with a 15-ft. taper to fit the existing sewer. A similar method was adopted at the baggage subway, the sewer being somewhat larger at this point and the dimensional details being different.

As to drainage, each subway is cut in two by the sewer, which is, of course, too high to permit drainage into it. Except at the stairway openings, the walls of the passenger subway will be lined with seats, and under each seat will be an open heating duct 12-in. wide and 16-in. deep. Under the stairway approaches the ducts are roofed with 4 in. of concrete, forming a 12 x 12-in. box. A larger duct connects the two at the station end and leads to the power plant. These ducts are sloped slightly each way to a succession of short laterals which lead to 4-in. cast-iron drain pipes laid parallel to the ducts. Other laterals lead from small covered sumps at the foot of each flight of stairs and at intervals in the subway floor. The drains in turn empty into concrete-lined steel manholes, which will be pumped out by automatic sewage lifts. These manholes are centrally located alongside stairways, at the edges of station platforms. Four of them are used, there being an independent drainage system for each side of the sewer and each side of the subway. The concrete is carried down to embed the entire drainage system.

A similar system, but simpler, is used in the baggage subway. The floor is pitched slightly toward the side on which the elevator pits are located, and the water drains direct into the pits. From there it is conveyed by means of automatic sewage lifts to one of two manholes, one each side of the sewer.

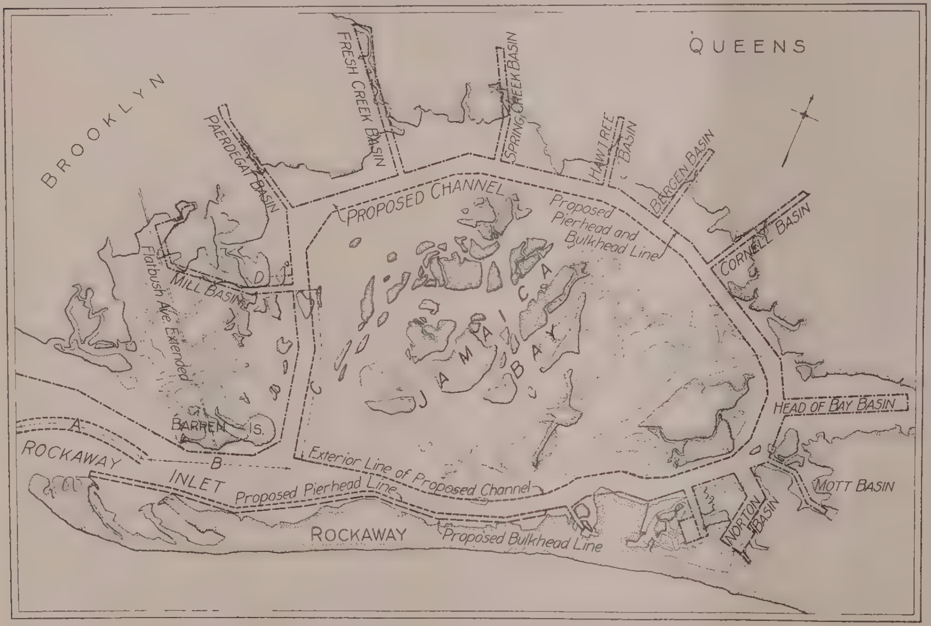
No drainage is provided for the short section of either subway over the sewer. The sewer happens to come immediately under tracks 1 and 2 for fast-freight service—the interior tracks of a group of four—and there are therefore no adjustments to be made to fit stairways or elevator lifts.

Henry W. Beebe, of Utica, is the contractor for the subways and platforms, J. Henry Miller, Inc., of Baltimore, having the contract for the station buildings. The work is being carried out under the general direction of G. W. Kittredge, chief engineer, and J. W. Pfau, engineer of construction. The subways were designed under the supervision of G. P. Stowitts, chief draftsman. G. F. Chism was until recently resident engineer on the work, reporting to E. B. Menuez, district engineer.

Jamaica Bay Improvement

Ship Channels and Piers Planned for a Minimum Cost of $75,000,000 for Barge Canal Terminus and New York Commerce

ABOUT $230,000 has already been expended on work now in progress on the improvement of Jamaica Bay, about 10 miles from City Hall, New York, for the construction of a harbor and establishment of a freight terminal at an estimated cost of $75,000,000, of which about 10 per cent will be borne by the United States Government and the remainder by the City of New York. The proposed improvement includes the development and maintenance of a deep-water entrance and channel through Jamaica Bay, the dredging of secondary channels and the construction of bulkheads, docks, piers and piersheds. The docks will be used as a terminal for the Barge Canal and for foreign and domestic commerce. Considerable work has been done by private capital. The Federal Government has approved the scheme of providing the deep-water entrance and has agreed to maintain it when completed.



Proposed Main and Secondary Channels for Jamaica Bay

Jamaica Bay occupies a large portion of the southeastern quarter of Greater New York and with its adjacent marsh land has an area of 45½ sq. miles. The shore line is about 25 miles and the average tidal range is 4½ ft. It lies close to the broad, deep water of the open bay and is thoroughly protected from storms by Rockaway Beach, which forms a natural breakwater. The New York Connecting Railroad, owned by the Pennsylvania and the New York, New Haven & Hartford Railroads, runs close to the bay, and the adjacent country is suitable for the development of large warehouse and manufacturing interests.

MAIN CHANNEL

The proposed main channel from the entrance at Rockaway Inlet to the head of the bay is about 20 miles, including both branches, and sections A, B and C, extending about 10 miles in from Rockaway Inlet, have already been improved sufficiently by the United States Government at a cost of about $150,000 to permit the navigation of vessels of 18 ft. draft. Section A, 18 ft. deep and 500 ft. wide, is now being dredged by the Federal Government. Section B has a natural depth sufficient for present needs. Section C was recently dredged by the United States Government to a depth of 18 ft. and a width of 500 ft. Sections A and B will eventually be dredged to a depth of 30 ft. and a width of 1500 ft., and the remainder of the main channel to a depth of 30 ft. and a width of 1000 ft.

AUXILIARY WORK

About 2,000,000 yd. of secondary channel and dock dredging have already been done by private capital, bulkheads have been built and ground filled in, thus constructing a large basin, which will be available as soon as the connecting channel D is dredged by New York City. The contract provides for the construction, if necessary, by the United States Government of two breakwaters to protect the entrance at an estimated cost of about $1,314,000 and for the maintenance of the main channel at an annual cost of about $55,000. To avoid complication, the dredging of the main channel, which is assumed by the United States Government, is executed by the City of New York, which is reimbursed by the Government at the rate of 8 cents per cubic yard up to a total of $4,720,000.

MUNICIPAL WORK INVOLVED

The proposed improvements to be effected by New York City when the project shall approach completion are estimated to include 60,000,000 yd. of dredging between the pier and bulkhead lines at 12 cents per yard, about $4,000,000 worth of bulkheads, 3000 acres of land at $1500 per acre, and 112 piers and piersheds.

The improvements already carried out by the City of New York consist of a channel dredged in 1913 from a point opposite the eastern end of Barren Island northward about 8000 ft. This channel is 500 ft. in width and 18 ft. deep. The material was handled by suction dredges and placed in a fill behind a light timber and sod bulkhead parallel with and 1100 ft. west of the channel, making an area of solid land about 7000 ft. in length and an average of about 700 ft. in width. The material was almost all fine white sand and was handled at a rate of about 500 yd. per hour by the larger of the two dredges, which has a 30-in. suction and a 27-in. discharge pipe; 1,733,785 yd. were dredged at a cost of $0.0749 per yd., including the construction of the bulkhead, making a total cost of $129,860.50. The entire cost was reimbursed to the city by the Federal Government.

The work was done by contract under the supervision of the Department of Docks and Ferries, of which R. A. C. Smith is commissioner; Charles W. Staniford, chief engineer, and C. E. Trout, assistant engineer in charge of dredging. The contractor was the Atlantic, Gulf & Pacific Company.

Concrete in Road Construction

Abstracts of Some of the Papers Presented at the National Conference on Concrete Road Building, Held in Chicago February 12 to 14

Concrete Road Experience in Ohio

By H. D. BRUNING
Division Engineer, Ohio State Highway Department

EXAMINATIONS of the twenty-nine concrete roads, totalling 39.63 miles, that have been built by the Ohio State highway department since 1910 have led to the following conclusions: Intelligent inspection and the use of only first-class material are essential to insure permanency; bank-run gravel or crusher-run limestone should never be used; a rich mixture is necessary as a factor of safety against irregularities in the mixing and placing of concrete; the wearing quality of the pavement is often impaired by insufficient protection against too rapid drying; there is no apparent difference in the permanency of pavements constructed on flat or crowned sub-grades; the weakest points are at the expansion-contraction joints; tarred felt is preferable to a poured joint; the shoulder should be reinforced by stone or gravel macadam for 1 to 2 ft. next to the pavement; a bituminous surface treatment is not essential; there is no special value in roughening the surface of the concrete preparatory to putting on the surface treatment; asphalts used as a surface treatment for concrete have peeled more than tars; and, finally, first-class concrete pavements have every indication of ability to stand up under heavy steel tire traffic.

FAILURE OF BITUMINOUS CARPETS

The results thus far obtained in applying a bituminous mat to twenty-three roads have rather discouraged further general application. The method follows: After the concrete is well hardened (not less than 10 days after placing) tar or asphalt heated to from 200 to 250 deg. Fahr. is applied, ½ gal. per square yard, and spread uniformly over the surface. While the tar is still soft there is applied a layer of coarse sand or pea gravel in the proportion 1 cu. yd. to 100 sq. yd. of surface. This forms a mat varying in thickness from ⅛ to ¼ in. Present specifications limit the covering to tar products and gravel, ranging in size from ⅛ to ¼ in.

The data obtained in an inspection just made by the speaker and Harwood Lersch, division engineer, show that on only four of the twenty-three roads has the bitumen given good results; on all others it has peeled off in varying degrees. An examination of the tar mat that peels off almost universally shows that adhering to its under surface is a film of inert mortar, from 1/32 to ⅛ in. in thickness. It thus seems that the peeling is not due to failure of the adhesion of the tar to the concrete, but either to an action of some ingredient in the tar on the concrete surface or, more likely, to the nature of the concrete itself, which makes it possible for the tar to pick up the top film of the concrete. Where the treatment is an oil asphalt, this condition is not true, and it is the experience of the department that such a product cannot be successfully made to adhere to the concrete.

Examination of the various roads reveals that tar mat filled with sand wears off much faster than one filled with pea gravel, which will resist abrasion to a higher degree. The treatment of tar and sand on Road 11, Youngstown-New Bedford Road, indicates a probable life not to exceed two years, having worn down to a thin mat not exceeding 1/16 in. after a use of only seven months under heavy mixed traffic. At a cost of 9.5 cents per square yard, an average cost on all roads treated, the maintenance would be rather excessive.

An interesting fact has revealed itself on Section 19 of the South High Street experimental road at Columbus, constructed under force account by the maintenance bureau of the department. A Dolarway treatment was applied at the rate of 0.36 gal. per square yard, and covered with coarse sand. A 33-ft. strip at the south end of this section was given two applications of the tar and sand. The part given one application peeled badly, while the other is in good condition after fifteen months' use under very heavy mixed traffic. On Section 20 of this same road, where Tarvia was used, what peeling has taken place has been on the parts which received the least tar.

TAR FOR SMALL REPAIRS

Owing to the apparent short life of the bituminous wearing course as heretofore applied, it would appear uneconomical to apply or maintain on the pavements thus far treated. However, the use of tar for treating cracks which develop in the pavement and of tar and mineral aggregate for patching defects such as worn joints and holes is very effective, and apparently the cheapest method of maintaining a good surface on such a road. Such repairs should be made at least once a year. Cracks, joints, or depressions should be swept clean of all foreign matter before the heated tar is applied. The mineral aggregate should be clean, thoroughly dry, and in size practically as large as the depth of the hole. If fine material is used the traffic will drive the tar to the top. This tar concrete should be thoroughly tamped. While asphalt may be used for filling joints and cracks, it is usually advisable to repair depressions in the surface at the same time, and for this latter purpose tar has given the best results.

The average cost of the concrete pavements completed to date is $1.128 per square yard. This cost is the contract price paid and does not cover engineering and inspection, but does include the cost of grading and finishing of shoulders and ditches and a small amount of accessories. Deducting from this average cost the average cost of surface treatment, which is 9.5 cents per square yard, there remains $1.033 as the average cost per square yard of plain concrete, including grading, shoulders and accessories. The average cost of grading, finishing of shoulders, and ditches and accessories is $0.167 per square yard of paved surface. Deducting this from $1.033 we have 86.6 cents as the average cost per square yard of the plain pavement alone, exclusive of engineering and inspection costs.

Experimental Concrete Roads

By JAMES T. VOSHELL
Senior Highway Engineer, Office of Public Roads, Washington, D. C.

PRIOR to 1911, when Congress initiated the practice of making annual appropriations financing road investigations, the Office of Public Roads was only able to furnish laboratory aid and engineering to such communities as asked co-operation. Naturally study of the early experiments are valuable only as showing the steps by which progress was attained. In Ithaca, N. Y., the boroughs of Richmond and Queens, New York City, and Bergen County, N. J., concrete test roads were built. At Ithaca in 1909 a 4-in. roadbed was covered with oil asphalt—penetration 16.8 mm at 25 deg. C.—in the proportion 1 gal. per square yard. Stone chips were added. By the following spring this coat was nearly worn away and subsequent applications of various bitumens have met the same fate. However, the concrete still receives an annual treatment. No transverse cracks have appeared and the pavement is in fair condition.

MIXING WITH PETROLEUM

During 1909 the Office of Public Roads conducted a series of laboratory experiments to determine the effect of mixing with ordinary concrete a residual petroleum at the rate of from 3 to 5 quarts per bag of cement. It was found that the strength of the concrete was not materially impaired, that it absorbed much less water and that the early shrinkage due to setting was much reduced. Experiments were conducted during 1910 and 1911 to test this material. Progress was made in the districts noted, but as the results are not striking attention will now be given to the experimental concrete sections in the Chevy Chase district, near Washington, built under the more generous Congressional subsidy.

A series of experiments was carried out in Montgomery County, Maryland, in 1912, to determine the relative value of bituminous concrete, cement concrete and brick-road surfaces when laid according to the best modern practice. These experimental sections, which had a total length of 6195 ft., were laid on Kensington Road beginning at Bradley Lane and extending to Chevy Chase Lake. The road has a double-track street-car line down the center of it, and the section paved is a strip 20.5 ft. in width on the west side of the street-car line.

In building the concrete section, 3950 ft. in length, it was purposed to determine not only the relative value of concrete as a material for surfacing roads but the relative values of plain concrete and oil-cement concrete, the relative values of gravel, crushed limestone and crushed trap rock for coarse aggregate, whether or not expansion joints were necessary, and, if so, the proper spacing, and the relative values of various bituminous materials when used for a carpet or paint coat. The required sections were built without joints, except the construction joints.

The plain concrete was a 1:1¾:3 mix. This was supplemented by a residual petroleum oil in the proportion 5 pints per bag of cement to make the oil-cement concrete. The coarse aggregates ranged from ¼ to 1½ in. in size. The sand was a good product obtained from the Potomac River. The concrete was mixed and placed by one machine, and was "struck off" and floated 6 in. thick.

To determine the changes in length of any section of the concrete road due to changes of temperature, brass plugs were embedded in the concrete, just before the final floating, in two rows 5 ft. from the edges of the pavement—one row with a 10-ft. spacing and the other with a 50-ft. spacing (see Engineering Record of July 19, 1913, page 76).

When the concrete had partly set it was covered with paraffined canvas, which remained in place until the concrete was hard enough for the surface not to be damaged by men walking upon it. The canvas was then removed and the concrete covered with a 2-in. layer of sand or earth, which was kept wet for a period of ten days. The construction joints were made by setting a header perpendicular to the subgrade and at an angle of 80 deg. with the center line of the road.

Occurrence of Cracks

When the sprinkling was discontinued and the concrete began to dry out, transverse cracks began to appear. The cracks did not appear at regular intervals, and their location was undoubtedly somewhat controlled by joints in a concrete gutter against which the pavement was laid. By the time the concrete had thoroughly dried out the cracks were an average distance of about 75 ft. apart. A few more appeared during the following winter, since which time none have developed. At present the average length of the monolithic sections is 43 ft. where gravel was the coarse aggregate and 121 ft. where crushed stone was used. The shortest monolith is in a gravel section and is 15 ft. in length, while the longest is a 220-ft. strip in the crushed-stone section.

Measurements have been taken from time to time between the brass plugs with a 10-ft. strain gage measuring to .001 in., and it has been found that the change in length per change of temperature, is, closely, that obtained by applying the factor for the coefficient of expansion of concrete and making a reasonable allowance for friction on the subgrade, showing that the concrete contracts more at the time of setting than it expands due to subsequent rises in temperature.

Of course the width of the cracks changes as changes in temperature occur, but ordinarily they appear to be from 1/16 to ⅛ in. in width. The largest cracks occur between the ends of the longest monolithic sections; those occurring at the ends of the 220-ft. monolith have an average width of about ¼ in. The maximum and minimum temperatures of the pavement so far observed were respectively 115 deg. and 25 deg. Fahr.; and the pavement shows no signs of having been damaged by this variation.

The edges of the cracks have been broken off to some extent by traffic, though not to such an amount as to be of much damage to the pavement. Those in the sections in which gravel was used as a coarse aggregate have broken off to a much greater extent than those in the crushed-rock sections, and all have broken off more than the edges of the construction joints in the same sections.

Tests of Surfacings

Last May a refined coal-tar, a refined water-gas tar preparation, a fluxed native asphalt and an oil asphalt were respectively applied to different sections of the plain and oil-cement concrete laid the previous November. The surface of the concrete was thoroughly cleaned by sweeping and washing and the bituminous materials were applied hot by means of a hand distributor at the rate of ½ gal. per square yard, after which a layer of stone chips or ⅜-in. gravel was spread over the surface and rolled with a 5-ton tandem roller.

In addition to the eight sections above mentioned, bituminous materials were applied to two other sections of the concrete surface. The sections were cleaned as above described and the surface of one was painted with an emulsified native asphalt at the rate of about 1/10 gal. per square yard and the other with a light refined water-gas tar at the rate of about 1/5 gal. per square yard, after which a fluxed native asphalt was applied as above described at the rate of about ½ gal. per square yard.

At the present time there is no apparent difference between the corresponding sections applied to the surfaces of the plain concrete and the oil-cement concrete. There is, however, a difference between the sections coated with the different bituminous materials. The sections to which the refined water-gas tar preparation was applied are in very good condition, as there are relatively few bare areas, the bituminous material is flexible and adhesion between it and the concrete surface is good. The refined coal-tar sections are in fair condition, but there are numerous small bare areas, and while adhesion between the tar mat and the concrete surface is good the tar appears to be rather "dead." The sections treated with the fluxed native asphalt and the oil asphalt are in good condition on the eastern two-thirds of the road, but on the western third there are a number of rather large bare areas. The asphalts are flexible, but do not appear to adhere well to the concrete surface. The section to which a paint coat of light refined water-gas tar was applied is in good condition—the bituminous materials are flexible, adhesion with the concrete surface is good and few bare areas occur. The section to which a paint coat of emulsified asphalt was applied is in fair condition on the eastern half of the road, while on the western half about 40 per cent of the concrete surface is bare. Adhesion between the asphalt mat and the paint coat appears to be poor.

A section 50 ft. in length was treated with a crude water-gas tar at the rate of about 1/10 gal. per square yard. This penetrated the concrete to such an extent that the surface at the present time has a dark-brown color. There was not sufficient material to form a mat, and it does not appear that this treatment is of any value in protecting the concrete surface from wear.

Conclusions

From the experiments conducted by the Office of Public Roads the following conclusions appear to be justified: The surface of the road should be smooth and uniform, requiring rather wet concrete. The coarse aggregate should be relatively small, preferably one in which all of the particles will pass a 1¼-in. screen. The concrete should be rich in cement in order that the mortar may be sufficiently strong and tough to resist the wear of traffic and to hold the particles of the coarse aggregate in place. Transverse contraction joints should be constructed, but expansion joints seem unnecessary. Particular attention should be given to the curing of the concrete by covering it and keeping it wet to insure that the mortar will have the essential qualifications of strength, hardness, and toughness, and that sufficient tensile strength may be developed, before initial shrinkage occurs, to prevent cracks from being formed between joints. These conclusions are only qualitative. It is hoped, however, that the experiments may eventually furnish sufficient data to warrant, at least, a few definite quantitative conclusions.

Financing Permanent Roads

By S. E. BRADT
Secretary, Illinois State Highway Commission

THERE are several sources of taxation from which to secure funds for road building—the Federal Government, deriving its income mainly through customs and internal revenue, thus drawing indirectly from special classes; the State tax, reaching all property within the State and including all the large cities and corporations, as well as many fees and special taxes; the county and township taxes, more localized in their scope and nearer the source of the benefits; and the automobile and kindred license fees, collected from a certain part of those benefited. By utilizing all of these channels practically all of the people benefited will bear a part of the expense.

The Government from its foundation has recognized the principle of aiding and fostering internal improvements. The reason given by legislators for not having already undertaken to assist in the road-building movement is the vast amount of money required. They claim that they dare not commit the Government to such a stupendous project. Some additional excuse for this backwardness is found in the fact that traffic conditions are demanding an entire change in road construction, and in the further fact that the advocates of good roads have not been able to agree upon a standard of construction, the method by which the Government should enter into the project or the amount of money to which it should be committed.

Broad Basis for Federal Aid

That the people are demanding action from Congress is apparent from the different measures recently introduced calling for appropriations for this work. Many of the bills introduced have failed to pass because they have asked the Government to build certain local and specific highways benefiting a comparatively small number of people. It is conceded that Federal aid should be upon a basis broad enough to reach every State in the Union and should be given only under such restrictions as would guarantee that the money would not be spent for temporary work, but would be employed in such construction as would endure. The people of the country are only waiting for a reasonable measure of assistance, and if the Federal Government will speedily develop a definite plan for such assistance it will give the needed impetus to start the work in all sections of the country.

The Government will soon reach the end of its expenditure for the Panama Canal. It seems appropriate that this fund, together with such other moneys as can be spared from internal improvements, equaling at least $50,000,000 a year for a period of fifteen years, should be diverted to road work. It has been very properly suggested and approved by all our National road organizations that Congress authorize the President to appoint a commission to investigate carefully the subject and recommend how and to what extent it shall enter into this work.

Serial Bonds

In order to construct an adequate system of roads we must utilize all channels of taxation. If we are to construct this system of main roads within a comparatively short period of time the tax should be spread over a period of years approximately equivalent to the life of the improvement. This can be done only through a bond issue. In the issuance of bonds for the public improvement not only should the maturity of the bonds fall within the life of the improvement but the maturities should be so arranged that the taxpayers would get value received each year from the amount of bonds retired during that year, thus requiring a serial bond. If put upon this basis, it is evident that a bond issue is the most equitable method of financing road construction.

Another feature that should be given careful consideration in connection with financing road improvement is the more economical expenditure of the funds that we are raising to-day for road purposes.

The Office of Public Roads at Washington estimates that we are raising at the present time for road and bridge purposes approximately $180,000,000 annually. In twenty years at this rate we should have raised $3,600,000,000, a sum large enough, if expended systematically by competent road builders, to give us from 300,000 to 350,000 miles of permanent roads, which with the present improved mileage would accommodate 90 per cent. of the traffic.

Township Tax

The $7,000,000 expended annually in Illinois is raised through a township tax by townships. Each township is divided into three road districts and elects a highway commissioner for each district, who has the expenditure of his share of the road money, averaging $1500, entirely independent of any other official. What railroad having $7,000,000 to expend in construction and maintenance would divide it among its section foremen and permit each foreman to build and maintain his particular section as he saw fit, regardless of any advice from the chief engineer, or regardless of what the other section men were doing? Yet that is similar to the system of highway management under which many States are operating.

County Units

The question is, How can we correct this situation and conserve this money for permanent work? My suggestion is that if we made the county the unit for road taxation and control of road funds, and put the care of the roads and bridges, as well as their improvement, under a county board of three members, one of whom should have had some technical knowledge of road construction and all of whom would give their time to the work, at the same time relying upon the State Highway Department for assistance and advice, we could place our roadbuilding upon a practical basis. This system would produce a marked saving in all lines of expenditure.

There still remains one channel of taxation—the automobile and kindred license fees. This special tax is levied because of the excessive damage to the roads by the motor-driven vehicles. These fees should, therefore, go to the maintenance fund, and if we should construct the more permanent type of road, such as brick or concrete, these fees will undoubtedly furnish an ample maintenance fund for years to come. The owners of these vehicles should be very willing to pay increased fees where the money is being properly expended either for more permanent road construction or proper maintenance.

Need for United Effort

If we can unite the Federal Government, the State, the county and the township in this enterprise under a conservative and carefully worked-out plan, we can finance the improvement of our main roads within a reasonable period of time without inflicting any great burden upon the people. In many sections of the country the people are demanding that this work shall begin without further delay and be carried on to an extent that will give promise of its early accomplishment. They want the roads now while they are here to use them, and if the present leaders do not provide the plan and the means by which the roads can be constructed the people will provide other leadership that will give them what they demand.

American Waterworks Association Analyzing Uses of Water

SEPARATION of domestic and non-domestic uses of water in cities is the subject of an investigation by a committee of the American Waterworks Association. It has long been recognized that the ordinary per capita consumption reported is not a fair basis of comparison because of the wide variation in manufacturing and industrial uses. For non-domestic use the committee wishes the data segregated under three heads—mills, factories, railroads and industrial plants; commercial use, such as stores, saloons, offices and the like; public use, street sprinkling, sewer flushing, public schools and schools. Information should be sent directly to E. S. Cole, chairman, Water Consumption Committee, 220 Broadway, New York City.

Of 16,600,000 Hp Available in Canada for the development of hydroelectric power, but 1,016,521 has been harnessed.

Experts to Redefine Sanitary Policy of Chicago

$10,000 Placed at Disposal of Committee to Study Local Situation

REALIZING that the Chicago main drainage canal is fast reaching the limit of its capacity, the Chicago Real Estate Board proposes to bring three outside experts, having no preconceived ideas in the sanitary problems of Chicago, to define anew the policy to be followed by the Sanitary District.

The sum of $10,000 has been placed at the disposal of the committee on harbor and river improvement of the Real Estate Board by private subscription, as the members of the committee, of which John W. Alvord is the engineer member, believe that a review of the sanitary situation and a study and report would be of the utmost value to the city at the present time. For this purpose it is planned to co-operate with the Sanitary District, so that the experts may quickly learn of the local situation through such data, statistics, maps and analyses as have already been collected.

Future of Sanitary Policy

Twelve problems were suggested as necessary to outline the future sanitary policy. Briefly, they relate to the dilution factor, with and without separation of trade waters and solids; to the degree of sewage treatment necessary; to the number and location of plants; to the question of extending branch dilution channels in place of interceptors; to the sanitary limits of dumping in the lake; to the problem of abating local nuisances in the three branches of the Chicago River; to the handling of the trade wastes; to the necessary plant, time and cost of its construction, should the dilution water be limited to 10,000 sec.-ft.; to what extent should reliance be had for a pure-water supply upon the preventing of accidental pollution of the lake by steamships, or from the deposit of dredgings, and when should reliance be had on the purification of the water supplies by filtration, sterilization, or other purification methods; to the possibility of contamination from future lakefront harbors and the necessary preventive measures to be adopted.

Limited Capacity of Channel

A former report to the board stated that from 7000 to 8000 sec.-ft. are at most times withdrawn from Lake Michigan, while only 10,000 sec.-ft., the amount allowed by the International Waterway Commission, are all that can be hoped for under the most favorable condition; further, that the channel, now thirteen years in use, has reached the limits of its capacity for the present population; that the district has been extended to the south and north far beyond that for which the channel was originally designed, a feature materially adding to the embarrassments under which the works now labor.

Negotiations have been undertaken with two prominent English engineers and one American engineer, but final employment will not be decided upon until definite co-operative arrangements with the Sanitary District are perfected.

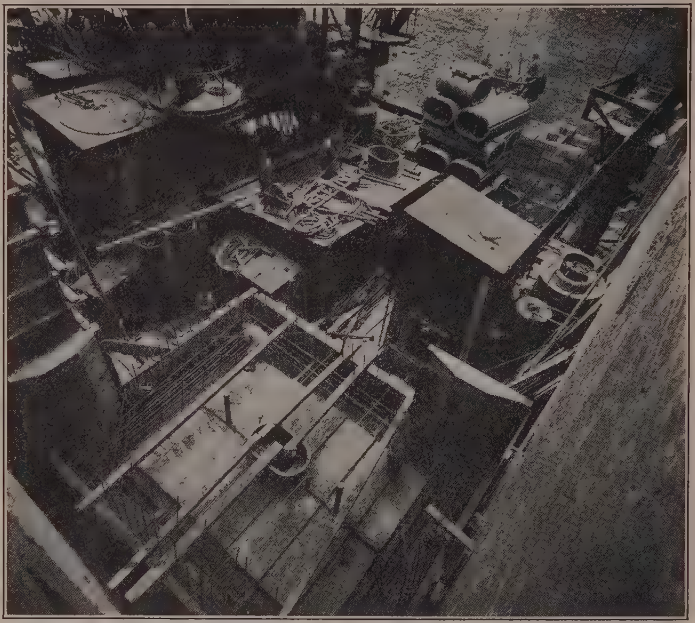
Six 3000-Hp Electric Furnaces are now in use by the Cyanide Company, of Niagara Falls, for the production of nitrogen ammoniate fertilizer, using current at about $15 per horsepower-year.

Pneumatic Caisson Foundations 100 Feet below Street Level

Concrete Working Chambers of Western Union Building, New York, Sunk in Limited Working Space

THE Western Union Building at 16 Dey Street, New York, is a tall office building, with its steel columns supported on concrete piers with foundations sunk by the pneumatic-caisson process to bedrock, about 100 ft. below the curb. The piers were made integral with their reinforced-concrete working chambers and were built to full height in the general excavation before sinking was commenced. The interior columns are supported on five circular piers and the wall columns on rectangular piers sunk close together in line and connected to form a continuous solid concrete wall inclosing the lot on the four sides, with the bottom sealed to the rock to exclude ground water. The connection between adjacent piers was made partly by vertical shafts between their opposite and adjoining faces and partly by tunneling under the cutting edges of two caissons, which were sunk simultaneously.

Sinking Large Caissons Adjacent to Existing Wall



Corner Caisson Reinforcement Ready for Concreting

WORKING CHAMBERS AND PIERS

The five interior piers are 10 and 11 ft. in diameter, and the exterior piers are all 6 ft. thick, except two that are 8 ft. thick and one larger corner caisson designed to carry the columns of the present building and the smokestack of a future adjacent building. The old buildings were removed and the excavations below the cellar floor carried 20 ft. below curb and the cutting edges of the caissons assembled in the bottom of the pit. They consisted of 12-in. channels on the flat, with flanges upstanding and having a 6 x 4-in. bottom angle riveted to the web.

The walls of the working chambers were reinforced with deformed bars and the working chambers and the piers above them were monolithic masses of concrete made in wooden forms with 2-in. tongue-and-groove sheeting 8 ft. long. The forms were made in the usual manner, with outside yokes of planks bolted together flatwise, and were removed, after the concrete was set, except on the sides in contact with the adjacent buildings, where they remained permanently in position because it was difficult to remove them and were sunk with the concrete.

Adjacent faces of the wall caissons were spaced 18 in. apart. The forms for the circular piers were made of 12-ft. length of 3-in. lagging spliced to 3-in. scarf boards. Each caisson was provided with a collapsible oval steel shaft containing a ladder and a bucket well and surmounted by an air lock. After the caisson was sunk the shafts were collapsed and removed. Air pressure was delivered to the working chambers through vertical pipes permanently embedded in the concrete.

The concrete, made of Lehigh cement and mixed in a Smith machine, was handled in bottom-dump buckets, and for filling the working chambers and shafts it was chuted from the mixer to the center of the lot and thence run through 9-in. pipes to the bottoms of the piers. For the formwork it was handled by the derricks.

Ground water was encountered about 26 ft. below the curb. Excavation was carried through quicksand and gravel containing boulders up to 1½ yd. down to the surface of the hardpan, about 76 ft. below the curb, and through it to rock at a depth of about 100 ft. In some cases two adjacent caissons were sunk simultaneously, and when they reached the hardpan a passage was tunneled through it from one working chamber to the other and the remainder of the excavation carried down in a single pit for both caissons.

In most cases, however, the caissons were sunk alternately, and after both of them were down through the quicksand an excavation 2 ft. deep was made between them, a short section of airshaft concreted in it, an airlock placed on top, and pneumatic pressure provided, under which the shaft between the ends of the caissons was excavated and sheeted down to hardpan and thence carried without difficulty to rock and filled with concrete, making a solid key bonding the adjacent caissons together.

There was a total of about 5000 yd. of open excavation carried down to a depth of 50 ft. below the curb, besides which there were 7000 yd. of caisson excavation. There are 7000 yd. of concrete in the caissons and 2000 yd. of reinforced concrete in the lower floor, which was also made by the foundation contractors.

William Welles Bosworth is the architect of the building and the Foundation Company was the contractor for the foundation work, which was executed under the direction of C. S. Rindsfoos, secretary of the company.

Tension in Scow Mooring

Cable Pull, Due to Swift Current, Measured by Dynamometer and Checked by Formulae

THE erection of the Canadian-Pacific Railway bridge across the St. Lawrence River at Lachine, Canada, involved the launching into position of four 1300-ton, single-track, 408-ft. spans, the forward ends of which were supported on scows, while the rear ends traveled on rollers. The two scows, braced together, were moored by a heavy tackle to a bridle connecting them with anchorage 1500 ft. upstream. They were subjected to a current varying from 5 to 8 miles per hour, and as the span was launched to position the scows moved at right angles to the current and were made to travel in an approximately straight line, instead of in the arc of a circle, by continued adjustments of the mooring lines. A dynamometer inserted in the reaving of the tackle recorded the tension, which, it was stated, corresponded generally with Froube's formula $R = f_s \ (v/6.9)^2$, in which R is the resistance in pounds; s, the wetted surface in square feet; v, the current velocity in miles per hour, and f, an empirical constant, fixed by experiment at 0.925.

For this case s was 10,800 for a draft of 8½ ft.; v, 7.5 actual, and R was computed at 19,000 lb. The maximum reading of the dynamometer represented a total pull of 16,800 lb., and the average reading, 14,000 lb. The anchorage equipment was designed for a strain of 100,000 lb. The operations referred to are described in a paper on the St. Lawrence River Bridge presented to the Canadian Society of Civil Engineers by P. B. Motley, bridge engineer of the Canadian-Pacific Railway.

Seventy-two Inch Wood-Stave Pipe Carried on Longitudinal I-Beam Stringers

Power Development at Saxon Falls

Six-Foot Pipe Line and Surge Tank, Both of Wood Staves, Receive Water Impounded by Multiple-Arch Concrete Dam in Michigan

By C. A. ALDERMAN
Chief Engineer, Ironwood & Bessemer Railway & Light Company

A HYDROELECTRIC development of 1250 kw has recently been completed on the Montreal River, 4 miles above its mouth. The power plant is located 17 miles northwest of Ironwood, Mich., and 25 miles east of Ashland, Wis. This is the third water-power station put in operation by the Ironwood & Bessemer Railway & Light Company. The power is to be used for lighting and street-railway operation in Ironwood and Bessemer, Mich., and Ashland and Hurley, Wis. The first installation of the plant comprises two 1000-hp units, but the ultimate development will have double this capacity.

HYDROGRAPHIC CONDITIONS

The Montreal River, the boundary line between Michigan and Wisconsin, has a drainage area of 280 sq. miles above Saxon Falls, with a mean precipitation of from 32 to 40 in. per annum. At Saxon Falls the river has a drop of 64 ft. With a 26-ft. dam this fall, together with the rapids in the river above and below, gives a head of 135 ft. The dam is located 1900 ft. above the power house. During the last year the minimum discharge of the river has been as low as 50 sec.-ft. The maximum discharge has not been ascertained, but from flood marks and other data it has been assumed that flood waters should raise the river 6 ft. above low stage, and the dam was, therefore, designed to take care of a rise of 8 ft.

Both the Montreal River and its principal tributary, the Gogoshungen, or west branch, have sites suitable for storage reservoirs, and four such locations have been surveyed, one on the headwaters of the Montreal River and three on the Gogoshungen. About 25,000 acre-ft. will be available in the four storage reservoirs.

The company has steam-power plants aggregating 1500 kw, at Ashland, Ironwood and Bessemer, which were in operation before the water power plants were built. These stations are maintained as auxiliaries in case of breakdowns or other trouble at the water-power stations. Two of the plants are in the same building as the car barns; in the other is housed the principal substation. The cost of keeping fires banked under the boilers is, therefore, small.

DAM

The dam is 190 ft. long. Its site is at the head of the rapids and just below an enlargement of the river basin, where the banks are high and the stream is narrow; there is a fairly good sandstone foundation. The type decided on involves concrete piers 5 ft. wide, 44 ft. long and spaced 16 ft. on centers. The piers were designed for an ultimate height of 30 ft. Their front faces have been carried up only 18 ft. and made heavy enough to take care of the water pressure to that height. The piers were built of such shape that, when the dam is raised, they can be made longer and more massive. At present the space between piers is closed with timber stoplogs in five of the bays, the other four having vertical reinforced-concrete arches. One bay was built for a Tainter gate 11 ft. 7 in. wide by 17 ft. high. When the dam is raised it is intended either to build it of reinforced concrete or fill in between piers with vertical or inclined arches.

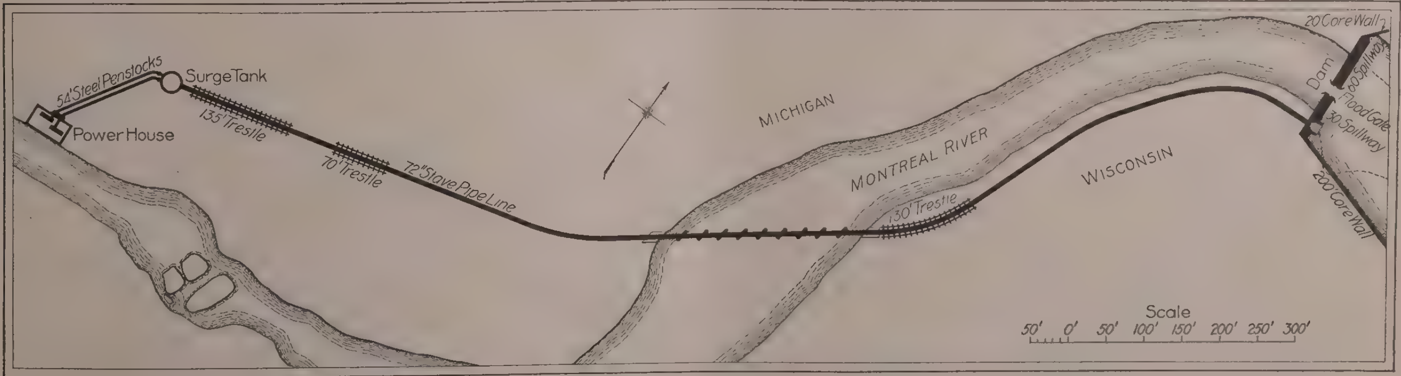
Underneath the dam a cutoff wall was sunk into the rock from 5 to 9 ft., according to the character of the stream bed. Holes 8 ft. in depth were put down on 4-ft. centers in the cutoff wall and the piers. In these holes were placed ⅞-in. square, twisted rods for anchorage. The concrete in the dam was placed from a concrete tower, 70 ft. high, located on the south side of the river. From this tower steel chutes conveyed the concrete to all parts of the dam. Gravel for concrete was obtained from the river bed and from a pit 2½ miles from the site, from where teams hitched to sleighs hauled 3 cu. yd. per trip.

The headworks are equipped with a gate 7 ft. wide by 7 ft. 10 in. high, operated by a gate hoist so adjusted that the gate is lowered by gravity. Two men can raise it in about ten minutes. The trash rack is made of 2 x ½-in. bars spaced 1½ in. apart. In order to avoid sharp curves in the pipe line the head gate is placed on the Wisconsin side of the river. The power house was built on the Michigan side because of a more protected location.

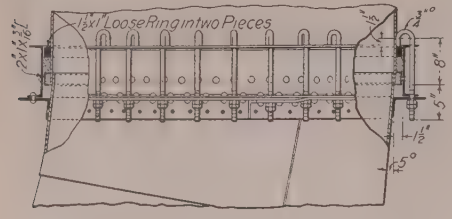
PIPE LINE

The pipe line is 1880 ft. long, made of 2½-in. hemlock staves cut to form a 72 in. diameter circle; 43 staves are required to form its circumference. The staves are cut to widths of 5⅝ in. outside dimension and 5¼ in. inside. A kerf, 1 in. deep, is sawn into the ends of the staves for 10-gage sheet-iron galvanized tongues. The pipe is banded with ¾-in. mild steel bands with malleable iron shoes, spaced from 5⅜ to 9 in. apart, depending upon the pressure, which varies from 8.84 to 15.75 lb. per square inch. Where the pipe rests on the ground timber saddles spaced 4 ft. apart are used to support it. These saddles also serve as forms for the construction of the pipe. The pipe is backfilled with earth to within 12 in. of its top. Where valleys are crossed the pipe rests on timber trestles composed of four 10 x 10-in. posts with cap and sill of the same dimensions. The bents are spaced 7 or 9 ft. on centers, according to their height. On these bents the pipe is supported by four 7 x 12-in. stringers.

Owing to the topography of the country it was necessary to place the headworks on the Wisconsin side of the river and the power house on the Michigan side, thus creating a river crossing. Nine concrete piers and two abutments, 3 ft. wide on top, 5 ft. on the bottom and 16 ft. in length, were spaced 26 ft. 7 in. apart across the river. Two lines of 15 in. I-beams support the pipe on these piers. The line is fairly



Relation of Dam, Pipe Line and Power House of Saxon Falls Hydroelectric Development



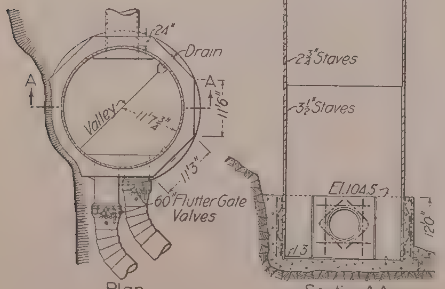
Detail of Pipe Expansion Joint

straight, with the exception of one curve of 128-ft. radius and a subtended angle of 36 deg. 40 min. near the headworks. It was, therefore, necessary to keep the hemlock staves in a steam box for about two days, after which period they were bent under pressure with a device rigged up on the ground.

Surge Tank and Penstocks

The 72-in. pipe line discharges into a 24-ft. diameter surge tank, 56 ft. high, built of hemlock staves sawed in a local mill. The lower 36 ft. are made of staves 3½ in. thick, the staves for the upper 20 ft. being 2¾ in. thick. One hundred and sixty staves were required to form the circle. A reinforced-concrete casing, 12 ft. high, is built around the base of the stand-pipe. Considerable attention was given to the design of the casing to make it strong enough to withstand the water pressure. One 72-in. pipe enters on one side and two 60-in. steel pipes leave from the other side. Below the hole formed by the entrance of the pipe line the casing is reinforced by twenty 1-in. square, twisted rods placed in horizontal rings. At the top of the casing above the pipes are placed sixteen rods. Between the upper and lower rings there are a sufficient number of trussed rods placed vertically to transmit the stress to these horizontal rings. The staves of the surge tank are banded with ⅞-in. mild steel bands spaced from 3½ to 7½ in. apart.

Dam at Intake, with Vertical Reinforced-Concrete Arches and Piers



Wood-Stave Surge Tank

The 54-in. steel penstocks are of 5/16-in. metal for the lower 60 ft. and of ¼-in. metal for the upper 57 ft. They are spaced 7½ ft. on centers. One of them was furnished by the waterwheel manufacturers and the other was built by a local boiler works. Reinforced-concrete anchorages, 6 ft. square and 6 ft. high, are placed underneath the penstocks and spaced 28 ft. 8½ in. on centers. Two ⅞-in. rods extend from 4 to 6 ft. from each anchorage into the rock. Penstocks are on a 40-deg. incline with the horizontal.

Each penstock is provided with a 60-in. flutter valve at the top of the pipe line. A 6-in. relief valve is placed in the penstocks adjacent to the flutter valves.

Power House

The power house is 30 x 53 ft. and is built of concrete on the Michigan side of the river, about 200 ft. below the falls. It was placed on this side of the river in order that it might be protected from floods, there being a projection of land into the river just above the power-house site. Both the power house and tailrace were excavated in the solid rock to a considerable depth.

The equipment consists of two horizontal 26-in., double-discharge Leffel waterwheels operating at 600 r.p.m. and using 83 sec.-ft. of water at a 135-ft. head to produce 1000 hp. Directly connected to the turbines are 625-kw, 2300-volt General Electric generators. The speed regulation of the units is accomplished by Lombard waterwheel governors. Two banks of three 200-kw, 60-cycle transformers step the current from 2300 to 33,000 volts. Two 14-kw, 125-volt exciters are belted to the generator shafts. One electrolytic lightning arrester, a switchboard and the necessary meters and recording apparatus complete the equipment.

It was decided to use this size of unit in order that full advantage could be taken of the stream flow, one unit being of the right size to take the minimum discharge of the river without storage and two units taking the discharge at mean ordinary stages. When storage reservoirs are built additional wheels may be installed of the same or larger size, depending upon whether a part or all the storage capacity is at first put into service.

Transmission Line

The company has approximately 83 miles of 33,000-volt and 20 miles of 15,000-volt transmission lines. These lines are carried on cedar poles from 30 to 60 ft. long, set from 110 to 150 ft. apart. They are cut from the woodlands adjacent to the line. Wooden cross-arms sawed in local mills are used with Pittsburgh High Voltage Insulator Company's three-petticoat porcelain insulators on steel pins. Galvanized steel gains are placed between poles and cross-arms. The Saxon Falls transmission line extends to Montreal, Wis., where it is tied in to another line extending from Ashland and through the Mellen power plant to Hurley, Ironwood and Bessemer. Two pole-top switches are used at Montreal in order that the Saxon or Mellen power lines may be cut out in case of line trouble at any point on either line. Other pole-top switches are used at the power houses in Ashland, Mellen and Ironwood.

The work was done with company forces by day labor, camps suitable for the accommodation of one hundred men being maintained at the plant. All material and equipment were hauled by teams about 3 miles over a road constructed especially for the installation.

Mechanical Filtration Plant for Toronto

New Works with Daily Capacity of 72,000,000 Gallons to Supplement Slow Sand Plant Filtering 57,600,000 Gallons

Bids have been received by the Department of Works of Toronto, Canada, for the construction of a mechanical water-filtration plant on ground adjoining the present slow sand plant on Toronto Island, a low gravel bar in Lake Ontario, 2 miles south of the city waterfront. The new plant is to supplement the one in use, and the fact that its capacity will be 72,000,000 U. S. gal. daily, as compared with 57,600,000 gal., which the slow sand filters were designed to supply, indicates the need the city has for extended purification facilities. It is required that the entire work be so carried out that the present plant shall not be harmed nor its operation interfered with.

Proposed Plant

The specifications state that the maximum required capacity for the proposed mechanical plant shall be 36,000,000 gal. during any ten hours when discharging into the present clear water reservoir (which covers 2.2 acres to a depth of 12 ft.) at the present level. The per day maximum, as stated, is twice this flow. The requisite bacterial removals are as follows: Ninety per cent when the raw-water content is 50 to 500 bacteria per cubic centimeter, 95 per cent when the limits are 500 and 2000, and 98 per cent when the count is more than 2000. A B. coli removal of 98 per cent, as determined by the standard methods of the American Public Health Association, is also insisted upon. Removal of turbidity must be accomplished by using not more than 1 grain of alum per imperial gallon (0.833 grain per U. S. gallon) with a minimum sedimentation period of three hours.

Coagulating basins, if included in the plan, and mixing tanks will be built of reinforced concrete. Any alternative for coagulating basins must be clearly demonstrated.

Filter tanks, which are to be in units of 4,800,000 U. S. gal., may be of steel or reinforced concrete. The filters will, as is usual, be grouped on either side of the pipe gallery, which will serve, as well, as an operating platform. Wash-water pumps, air blowers, chemical storage, and, if necessary, a clear-water basin complete the proposed equipment. The city will make the connection to the present reservoir, but the contractor must build 50 ft. of clear-water pipe.

ALTERNATIVES IN PUMPING EQUIPMENT

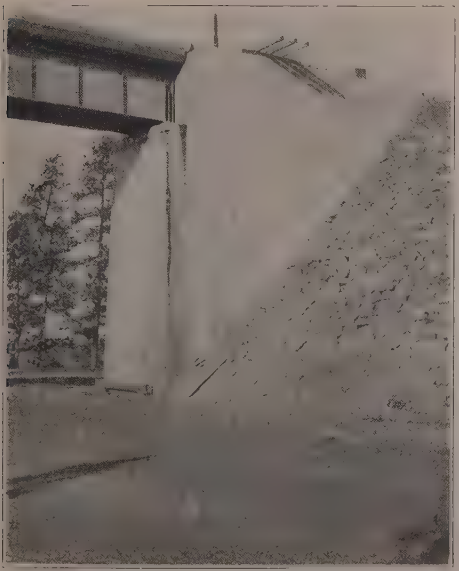
Low-lift pumping equipment will comprise three centrifugal pumps, each with a capacity of 43,200,000 gal. per day. While it is expected that these will be operated by 2200-volt induction motors, alternative proposal forms are offered covering vertical, compound or triple-expansion condensing engines, or other engines. Pumping capacities for the first type are to be determined by Venturi-meter measurement, when operated not faster than 350 r.p.m., with a piston travel not exceeding 80 ft. per minute under average conditions. These alternatives also apply to the wash-water and air systems.

The permissible seepage is that lowering the water level in any chamber less than ½ in. in twenty-four hours, after standing full for that period.

The rate controllers for the filters must maintain any desired flow between 3,600,000 and 72,000,000 gal. per day within a 5-per cent variation. The agitators in the solution tanks are to be driven by a 220 or 550-volt, 25-cycle electric motor. An overflow weir to prevent flooding the floor is to be placed in the pumping station 1 ft. below the elevation of the point of discharge into the clear-water reservoir.

It is desired that the first half of the plant be completed by Dec. 31, 1914. Indicative of the strong necessity for an increased supply of filtered water is the promise that for each day that this half is completed prior to Dec. 31 a bonus of $100 will be paid the contractor. Penalties for non-completion within the specified time are also specified—for the first half of the work, $200 per day up to 60 days and $300 per day beyond that limit, and $50 per day for the second half.

The four bids which have been received, from which no choice has been made, range from $1,096,277 to $1,733,463.75.



Old Pier Bonded to New One

Moving 300-Ton Bridge Pier

Massive Concrete Block Is Transported 60 Feet from Old to New Site

As a result of a change of plans in the location of a bridge over the Spokane River a 300-ton bridge pier of concrete was moved 60 ft. from its original position and used as part of an abutment at the new site. The North Coast Railroad Company, a subsidiary of the Oregon-Washington Railroad & Navigation Company, laid out in 1909 a single-track line on which it was proposed to cross the Spokane River over a steel viaduct at a point about 1½ miles below the city of Spokane. Construction on the proposed line was commenced by building on the north bank of the river the first two concrete piers for this viaduct. Before work proceeded further, however, the parent company entered into a contract with the Chicago, Milwaukee & St. Paul Railway Company for joint terminals in Spokane, and incidentally took over the construction of the North Coast line.

The location of freight yards on the north bank required a change in the proposed position of the north terminus of the viaduct over the river, and it was decided that one of the old piers already erected could be moved a distance of 60 ft. and there increased in size by the addition of new concrete so as to form the U-shaped abutment called for by the revised plans. The other pier, built according to original plans, could not be utilized in the new arrangement and was therefore destroyed by dynamite.

SCHEME OF TRANSPORTATION

The pier which was moved to a new location was 39.5 ft. high from base of foundation to top; its top section measured 12 x 5 ft., and the base, 16 ft. 3½ in. x 9 ft. 3½ in., the weight being about 300 tons. The pier was raised by 35 and 50-ton ratchet jacks, after which a bed of ties was laid to support nine 75-lb. rails, running lengthwise with the base of the pier. On these rails were placed twenty 5 x 5 in. steel rollers, and the weight of the pier was carried to these rollers through eight rails, of the same weight as the others, placed, flange up, against the base of the pier and parallel to the rails beneath the rollers. The ties were laid and the lower bed of rails was placed continuously to grade so that the pier could be rolled to its new location without shifting the upper bed of rails.

When the rollers were carrying the weight of the pier satisfactorily a tie was lashed to the lower bed and a 35-ton jack braced against it. This jack moved the pier on the rollers quite easily. The pier was moved at the rate of about 6 ft. per hour, and as it advanced the rollers were set at a slight angle so that it turned through the 30-deg. angle required to make it square up with the alignment in the new position. The set given the rollers was determined by taking, from time to time, the angle through which the pier turned in a given distance, and it was thus carefully brought to its new position. The twelve ratchet jacks were again placed under the pier and it was raised to allow removal of ties, rails and rollers, after which it was carefully lowered to the proper elevation. For this purpose the ratchet jacks were essential because of the impossibility of controlling the hydraulic jacks while lowering.

When the pier was brought to its proper alignment and elevation, one jack in the center of each side was removed and a concrete pedestal 2 ft. square was poured, rather wet, into rough forms and tamped well under the pier and run up on the side about 6 in. After allowing this to harden for three days the jacks on the corners were removed and the operation was repeated. When the last of the jacks had been removed a concrete base was poured inclosing the pedestals so as to form a solid new base.

Moving 300-Ton Pier

BONDING OLD WORK TO NEW

The old pier forms part of the front wall of the new U-shaped abutment, which is about 44 ft. high, with wing walls 36 ft. in length. After the old pier was successfully supported on its new foundation provision for bonding it to new concrete was made by drilling ¾-in. holes 24 in. deep on 2-ft. centers, and placing dowel pins in these before the new concrete was poured. About 1500 cu. yd. of new concrete were required to complete the abutment, and this was figured to cost about $9 per cubic yard. Figuring the concrete in the old pier as worth $9 per cubic yard in the new position there was thus effected, by moving the pier, a saving of $5 per cubic yard, or a total of about $940.

John D. Isaacs, Jr., assistant engineer of the Oregon-Washington Railroad & Navigation Company, planned the methods of moving the pier and personally supervised the work.

Removal of Concrete Forms

The committee on reinforced concrete of the Canadian Society of Civil Engineers has submitted to the society a draft of proposed standard specifications for plain and reinforced concrete. These specifications contain a clause entitled "Form Removal," which reads as follows:

"The forms shall not be removed until the times named in the following table have elapsed after depositing concrete, not counting periods in which the temperature has been below 35 deg. Fahr.:

Part	Minimum 24-hour day
Posts under beams and girders	20
Floor-slab panels	10
Wall forms	2
Column forms	4
Sides of beams and girders	4
All other parts	10

Economical Municipal Asphalt Repair Plant

Contract Unit Prices Halved during Second Year of Niagara Falls' Department

THE report for 1913 of F. S. Parkhurst, Jr., city engineer of Niagara Falls, N. Y., shows that the municipal asphalt repair plant during its second year of life yielded material to repair 17,364 sq. yd. of street, at a cost of 71 cents per square yard, 53 per cent of the price which formerly prevailed when the work was done by contract. The area cared for is more than eight times as large as that repaired in 1911, the last year in which the work was contracted. The average force required to run the plant and make the repairs (which totaled $12,369.38) consisted of twenty-five men and two teams. The average daily payroll was $78.60. Although the season's work covered five months, there were only ninety-seven working days.

The area of asphalt topping actually laid is 17,363.69 sq. yd. This material was made in 3088 batches, the total volume being 35,512 cu. ft. Binder applied covered 1008 sq. yd. and was made in 168 batches. Binder and topping were laid to practically the same average thickness—2⅝ in.

The cost division for the repair work is given in Table 1.

TABLE 1—COSTS OF REPAIRING NIAGARA FALLS PAVEMENTS IN 1913

Material	Asphalt topping, total cost	Binder, total cost	Concrete foundation, total cost	Brick pavement, total cost	Total cost
Sand and gravel	$342.50	......	$366.50	$175.32	$884.32
Cement	184.59	......	486.50	281.47	952.56
Ground limestone	94.43	$5.07	......	......	99.50
Asphaltic cement	1,423.26	76.49	......	......	1,499.75
Coal and wood	712.71	38.30	......	......	751.01
Crushed stone	....	27.60	344.32	47.73	419.65
Bricks	....	......	......	219.53	219.53
Plant and tools	1,549.69	83.28	......	......	1,632.97
Miscellaneous	....	......	226.03	281.36	507.39
Total cost of material	$4,307.18	$230.74	$1,423.35	$1,005.41	$6,966.68
Teaming	1,347.03	72.39	236.10	300.13	1,955.65
Labor	6,070.28	326.22	1,477.00	2,001.10	9,874.60
Total	$11,724.49	$629.35	$3,136.45	$3,306.64	$18,796.93

Plant depreciation brings the total charge against asphalt topping and binder to $13,033.39. The inventory of the plant and equipment, taken Dec. 1, 1913, set the total value at about $1,000.

The average labor division for the season's work is given in Table 2, and Table 3 shows the saving to be accredited the municipal plant during its two years of use. The great increase in production since municipal control began has probably been an important factor in reducing the unit costs.

TABLE 2—LABOR SUBDIVISION AT MUNICIPAL ASPHALT REPAIR PLANT

At Plant

No.	Occupation	Rate per day	Total
1	Engineer	$5.00	$5.00
1	Foreman	3.50	3.50
2	Machine tenders	2.50	5.00
2	Ordinary laborers	2.00	4.00
6			$17.50

Skilled Street Work

No.	Occupation	Rate per day	Total
1	Superintendent	$6.00	$6.00
2	Rakers	3.50	7.00
1	Smoother	3.25	3.25
1	Tamper	2.75	2.75
1	Timekeeper	3.50	3.50
6			$22.50

Ordinary Street Work

No.	Occupation	Rate per day	Total
4	Choppers	$2.00	$8.00
2	Rollermen	2.00	4.00
1	Painter	2.00	2.00
6	Cleaning-up	2.00	12.00
2	Teams	6.30	12.60
15			$38.60

Average number of men, 25; average number of teams, 2; average daily pay roll, $78.60.

TABLE 3—COST COMPARISON OF PRIVATE AND MUNICIPAL WORK

	Date	Square yards repaired	Cost per square yard	Total cost
Private contractor	1907	1,839	$1.35	$2,482.65
Pricate contractor	1908	1,180.43	1.35	1,593.58
Private contractor	1909	1,509.22	1.35	2,037.45
Private contractor	1910	3,220.26	1.33	4,282.95
Private contractor	1911	2,091.17	1.35	2,821.73
Municipal plant	1912	12,243.9	.77	9,414.21
Municipal plant	1913	17,363.69	.71	12,369.38
Total	....	39,447.67	....	$35,001.95

Inland Waterway Terminals

Arguments for Co-operation between Railroads and Canals, with Adequate Transshipment Facilities for the Latter

BELIEVING that of every $1,000,000 expended on canals $750,000 should go for terminal facilities, Sterling Morton studied these points in a European trip last summer and has put his observations into a sixteen-page pamphlet, from which the following notes and conclusions as applied to this country are taken. Although advocating rehabilitation of the waterways of the United States, he is of the opinion that the present policy of bending all effort to channel betterment is wrong. Canals should be made at least 7 ft. deep and rivers dredged to that depth, but the main efforts should now be directed toward the installation of harbor works, handling apparatus and the construction of modern boats, so that the natural opportunities for commerce now existing may be employed.

The swift development of the railway systems has drawn attention from the waterways now existing and no apparent effort has been made to improve the boats, channels or handling facilities. The railroads of the United States are now under the control of the Interstate Commerce Commission and laws have been passed especially designed to prevent unfair competition with waterways. But the canals and rivers are no longer in use, as the fleets have vanished, the channels are neglected and no terminal facilities for water shipment exist, except in an antiquated form. Mr. Morton believes that the railroads themselves would be the greatest beneficiaries of a rehabilitation of the waterways and the diversion to them of a great deal of low-class, bulky freight. An English commission has proved that the greater part of the operating expense of the railways there is incurred in handling low-class freight, which produces the lesser part of the revenue. In this country periodic car shortages are experienced, at which times the movement of one of the commodities producing the lowest ton-mile revenue—coal—is of prime importance, and high-class articles must wait until the coal is moved. Imagine the relief to the Illinois railways were it possible for them to haul the coal to some point on the Illinois River for delivery by barge to Chicago and St. Louis. This would be possible were the 66 miles of old canal between Lockport and La Salle put in modern condition. Then the railroad terminals at these cities would be relieved of a huge amount of hardly profitable tonnage and the commodities producing large revenue could move freely, with plenty of equipment and motive power.

Co-operation between rail and water must come in this country as it has abroad. The railroads would reap the greatest benefit and should be in the position of advocates, instead of that of opponents, to any legitimate waterway projects. Germany has proved that the two can exist side by side to mutual benefit, and there is no reason to fear that the same would not be the case here.

COMPARISON WITH RAILROADS

The arguments against the use of waterways are mainly expense of construction and maintenance, slow movement of freight, limited area served, lack of handling facilities and low rail rates. It is, of course, cheaper to build and maintain a single-track railway than a canal, but a single-track line cannot attempt to handle the traffic that is possible on a canal even of moderate dimensions. The comparison is wrong. The yearly tonnage per mile handled on certain sections of the Rhine is fifteen or twenty times as great as the tonnage per mile of the German railways. A canal admitting a 600-ton barge and taking fifteen minutes to a lockage will handle 24,000 tons in ten hours, or the equivalent of 600 40-ton freight cars in the same period, or 1840 such cars in twenty-four hours. If a canal such as the Illinois and Michigan canal, only open 200 days in the year, is capable of taking the same size barge and operate full capacity sixteen hours a day, the yearly tonnage would be 7,680,000. No single-track railway could handle this.

On investigating the second objection it is found that the average movement of a loaded freight car, according to American Railway Association figures, is 25 miles per day. Sixteen-hour running at even 3 miles per hour on the canal would give a total mileage about double that. Recent experiences show that even on the antiquated and decadent canals of the United States water shipments arrive at destination in less than the average railway time.

AREA SERVED

The question of limited area served is merely one affecting the ultimate development. There is sufficient tonnage originating on our rivers for shipment to other points on the rivers to keep busy a fleet surpassing that of the Rhine. In addition to this, a system of equitable perpendicular rates would bring a huge volume of commodities to the waterways for a part of the longer hauls. Proper transfer facilities would make this advisable where the water could be used for 100 miles or more.

The handling facilities would soon be provided in the wake of any waterway development. Far-sighted cities and corporations would provide them in advance of development, as has been done in Germany, and reap rich rewards. It is beyond the bounds of possibility that the nation which has put in service the wonderful unloading machines seen on the Great Lakes would be backward in devising methods of handling other kinds of freight.

Rates by rail are indeed low in this

country, but this argument loses force when the actual figures are used instead of generalizations. The cost of transporting goods by water, in barges of 600 tons or over, cannot be more than 3 mills per ton-mile (probably much less), and at that price will pay dividends. The railroads cannot exist at such revenue.

Barges and tugs are not as costly as freight cars and engines. Modern steel barges can be built here for $10 per ton of carrying capacity. A steel towboat equipped with an internal-combustion engine, carrying only 20 tons less freight than the barge and capable of towing two barges, would cost $9,000. Thus a fleet to carry 1780 tons (the average train load) would cost $21,000—which is less than the cost of the locomotive of the freight train. A captain, engineer, cook and six deckhands could easily handle the fleet—more men, it is true, than the conductor, engineer, fireman and two brakemen of the freight train; but it must be remembered that the train requires telegraphers, switchmen, trackmen, towermen, roundhouse men, etc., for its operation, while the crew of the barge fleet is self-sufficient.

FUEL COSTS FOR TUGS

The fuel cost of the tug is much less than that of the engine. It has been determined that one horsepower draws six times as much on smooth water as on level rails. Allowing for propeller slip, etc., it is reasonable to suppose that a pound of coal will do at least three times more useful work in a tug than on rails. An internal-combustion engine burns approximately 1 pint of oil per horsepower-hour. A 50-hp engine would move the fleet described above at 4 miles per hour with ease. On kerosene at 6 cents per gallon the fuel would cost 38 cents an hour—an almost unbelievable figure.

Benefits of cheap water transportation involve three essential necessities: Co-operation between rail and water lines, the building of proper harbor works and installation of modern rapid and economical methods of handling cargo at both terminal and intermediate points on our waterways, and the rehabilitation and enlargement of existing canals to accommodate barges of at least 600-ton capacity and the early construction of new canals connecting the principal natural waterways.

Regulation of the streams Mr. Morton considers of secondary importance. The Mississippi valley now has 2500 miles of 6-ft. navigation, all connected, as compared with only 350 miles of similar depth on the Rhine. Terminal facilities are as essential on canals as on railways.

Car-Shop Column Piers and Service Tracks for Concreting Them

Additions to Elizabethport Shops of the Central Railroad of New Jersey

Large Steel Car Shop with Moderate Roof Spans and Uniform Bents; Planing Mill, Paint Shop and Sand-Blast House with Large Windows

THE shops of the Central Railroad Company of New Jersey at Elizabethport, N. J., are designed for the maintenance and repair of rolling stock for passenger and freight service. To the older buildings, which include a 25-stall roundhouse, a 160 x 700-ft. erecting and machine shop, a large car-repair shop and the usual accessories, have recently been added a second car-repair shop (a steel-frame building 600 x 180 ft.), an 81 x 300-ft. planing mill, an 81 x 200-ft. finished lumber storage building, a 31 x 92-ft. sand-blast house and a 53 x 200-ft. steel-roof paint shop. A feature of the new work, as the illustrations show, is the large amount of window space provided. Even in the car shop, in spite of its width, an unusual amount of daylight is admitted. In the side walls the panels between columns are almost entirely glazed except about 5 ft. next to the floor, and the interior is lighted by clerestory windows in the 52½-ft. aisle and by sawtooth roofs in the adjacent intermediate aisle.

CAR SHOP

Except for the office at one end, which has brick walls, the walls of the car shop are concrete, spanning from column pier to column pier independent of the ground, thus virtually forming a deep belt course of girders supporting the steel window frames.

The column piers were built on low ground and carried up several feet above the original surface to provide for the future higher grade. Their forms were concreted directly by dump cars on service tracks carried from the mixing plant on cribbing and trestles built up high enough to serve all the forms. After the completion of the piers the site was filled up to yard grade and the shop floor, consisting of 3-in. plank spiked in 3 x 10-in. creosoted joists, was laid on top of the cinder-and-earth fill.

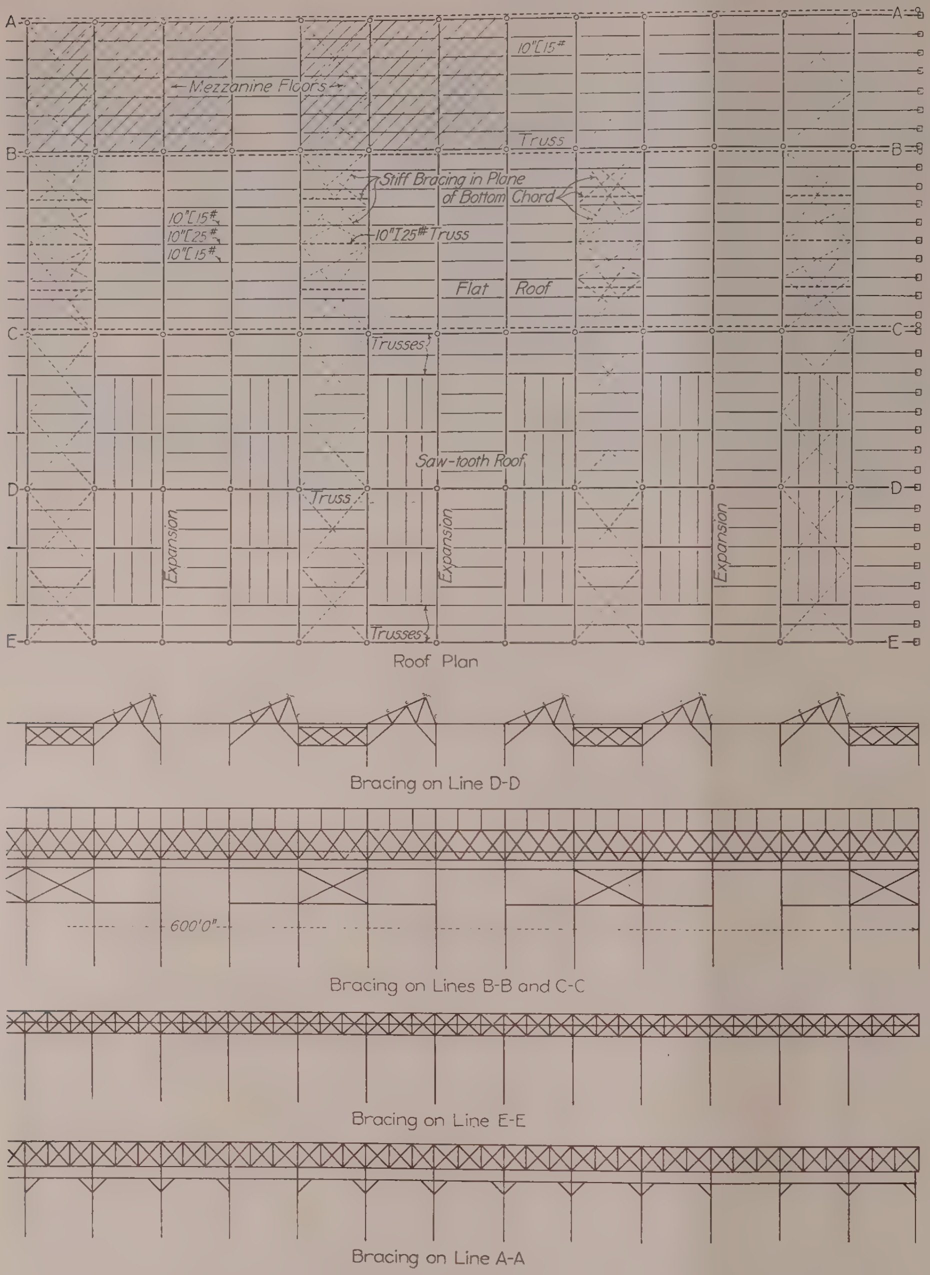
Five longitudinal rows of columns divide the building into aisles of 37½, 52½, 44¼ and 44 ft. on centers. The 52½-ft. interior aisle has a clear height of 40 ft. under the roof trusses and is lighted through clerestory windows above the roofs of the adjacent aisles. It is served by two 25-ton traveling cranes with full-length runways and is used chiefly for freight-car repairs. The flat roof is pitched slightly in both directions and is X-braced by 1⅛-in. rods in every third or fourth panel of the top chord and by stiff lateral X-bracing of the bottom chord.

The 44-ft. aisle has a minimum clearance of about 21½ ft. under the roof trusses, the top chords of which are pitched slightly downward to the outer wall, carry sawtooth trusses in alternate panels and are X-braced by 1⅛-in. rods every third or fourth panel of the top chord. The 37½-ft. aisle has in three places a full-width mezzanine floor 60 ft. long suspended from the roof trusses 15 ft. clear of the ground floor to provide space for toilets and heating systems.

UNIFORM BENTS

In the main framework there are twenty-nine transverse bents spaced 20 ft. apart, making with the end brick walls a total of thirty panels. The bents are substantially alike except for the slight differences occasioned by the mezzanine floors, and have five I-shaped columns extending to the roof line with gusset-plate connections to the top and bottom chords of the roof trusses.

Car Shop, Showing Yard Crane Girders and Large Vertical Windows



Part Diagrams of Steelwork, Showing Arrangement of Trusses and Bracing

They have five longitudinal lines of braces, one at the top of each row of columns. On both sides of the 37½ ft. and 52½-ft. aisles are continuous lines of plate-girder runways for the overhead cranes, and above them are light riveted trusses about 6½ and 8½ ft. deep.

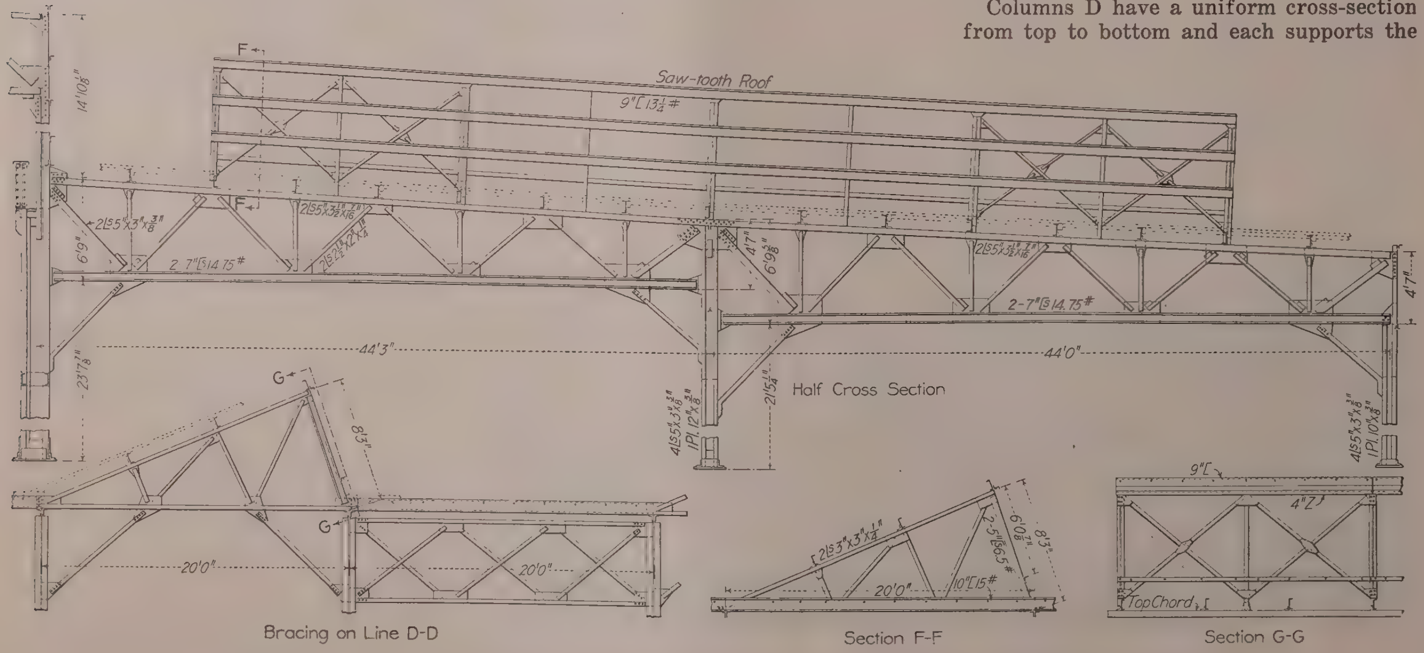
Between the two 44-ft. aisles the tops of the columns in the fourth panel are connected by light lattice trusses about 5½ ft. deep in panels alternating with sawtooth trusses except where expansions occur. These sawtooth trusses are knee-braced to the lattice trusses, providing bracing in sets of three panels separated by single unbraced panels. The columns in the outer wall of the 44-ft. aisle are braced together by riveted trusses 6 ft. deep. Provision for expansion is made about every 80 ft. by slotted-hole longitudinal connections.

LOADS AND STRESSES

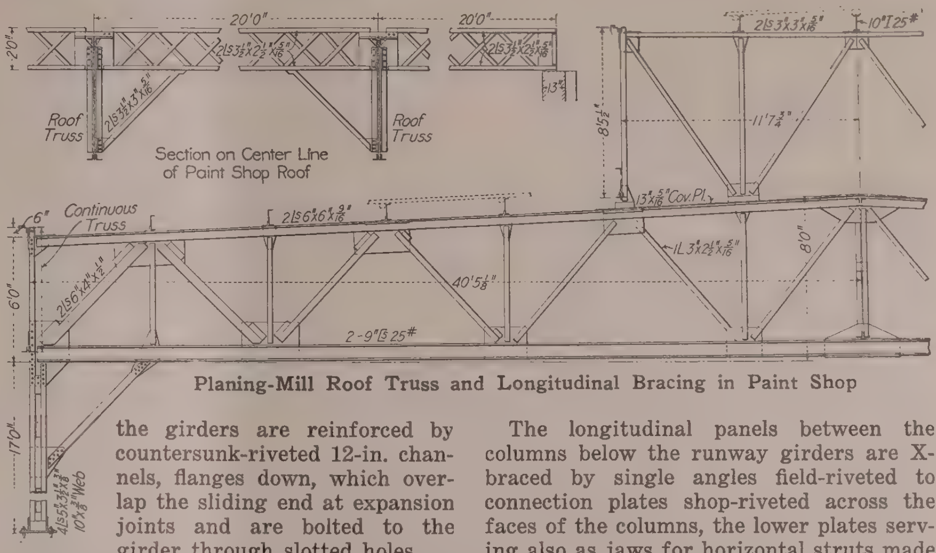
The top chords of all roof trusses carry channel-iron purlins sheathed with 2½-in. roof boards which are assumed to weigh 10 lb. per square foot and are covered with tar, felt and gravel roofing weighing 5 lb. per square foot. The steel work is estimated at 12 lb. per square foot and live load is assumed at 50 lb., making a total purlin load of 65 lb. per square foot and a truss load of 77 lb. per square foot. The mezzanine floors have a 4-in. concrete slab weighing 46 lb. per square foot, finished with a 1-in. cement surface weighing 12 lb. and the beams and protecting concrete weigh 25 lb. per square foot, making with the 100-lb. live load assumed a total floor-beam load of 183 lb. per square foot.

The runway girders are proportioned for two 25-ton cranes coupled, with 10-ft. wheel-bases 13 ft. apart on centers and a maximum wheel load of 40,500 lb. All columns are considered to have fixed ends top and bottom. Referring to the accompanying roof plan, the A columns have a maximum reaction of 213,000 lb. and a bending moment of 828,000 in. lb.; the B columns, a reaction of 254,000 lb. and a bending moment of 828,000 in. lb.; the C columns, 188,000 lb. and 828,000 in.-lb., respectively; the D columns, 72,000 lb. and 403,200 in. lb., and the E columns, 38,000 lb. and 400,000 in. lb.

Columns D have a uniform cross-section from top to bottom and each supports the



Details of Roof for Car-Repair Shop, Showing Design of Sawtooth Rafters and Supporting Trusses



Planing-Mill Roof Truss and Longitudinal Bracing in Paint Shop

adjacent roof trusses by field-riveted connections of their top chords to a section of web plate riveted between the flange angles at the top of the column and projecting beyond them on both sides. Columns E are similar to columns D except that their cross-sections are lighter and that the inner pairs of flange angles and the webs terminate at the bottom chord of the roof truss, above which the outer flange angles extend to receive the field-riveted connections to the end gusset plates of the top chord of the roof truss.

All other columns have upper and lower shafts, the former receiving the load from the roof trusses, while the latter receive both the roof and the crane-girder loads. The lower shafts have an unsymmetrical I-shaped cross-section with a deep solid web plate and one flange reinforced by a channel; the vertical flanges of the channel are turned in and proportioned to take the load from the runway girders seated on a cap plate directly over the center of the reinforced flange. The other flange is extended above the runway girder with a much narrower web plate and a second flange spliced to the top of the web in the lower shaft. The inner flange of the upper shaft terminates at the bottom chord of the roof truss, to which it is field-riveted through the end gusset plates, and the outer flange is continued to receive the top chords of the roof trusses and of the longitudinal girders.

ROOF TRUSSES

The roof trusses over the crane girder in the 52½-ft. aisle have angle-iron bottom chords. All of the other roof trusses have bottom chords made of pairs of channels back to back, and all other truss members are made of pairs of angles back to back. Light diagonals form the web members of all trusses, with verticals from the bottom-chord panel points to support the top chord intermediately. Except in the 52½ ft. aisle, all trusses are knee-braced to the columns from the end bottom-chord panel points by pairs of angles back to back with the lower ends bent vertical and the upper ends engaging opposite sides of the projecting connection plates.

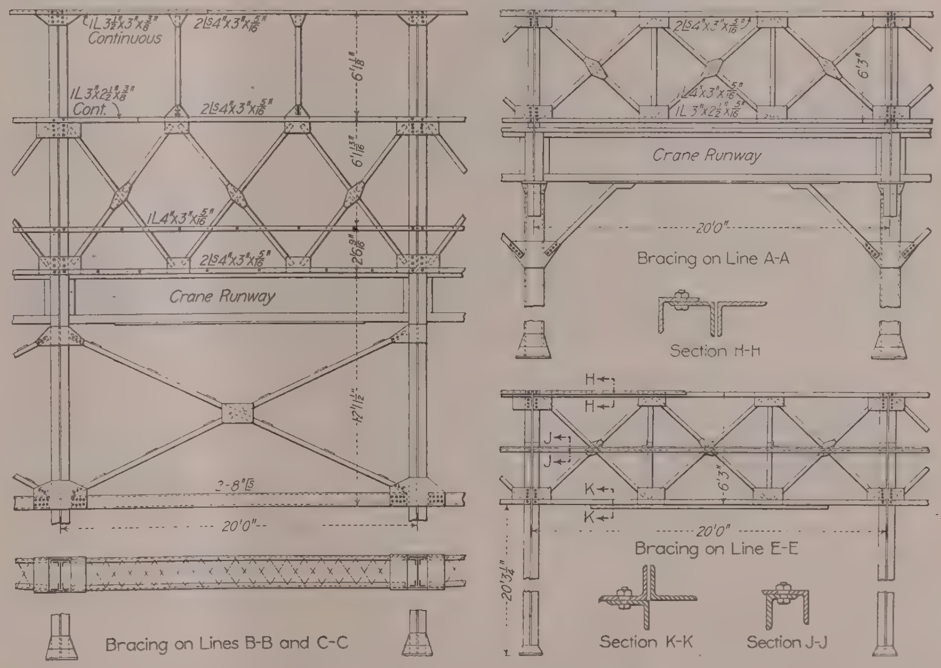
There are two lines of runway girders for the crane in the 52½-ft. aisle, and a third line is supported by the wall columns in the 37½-ft. aisle to carry an outside crane in the stock yard. The runways are 36-in. plate girders of 20-ft. span and have 7/16-in. web plates without stiffener angles except at the end bearings, where they are field riveted together through vertical transverse connection plates which form part of the webs of the upper shafts of the columns and project beyond the flanges of the opposite side to receive the field-riveted connections of the roof trusses.

At expansion points, 80 ft. apart, a clearance of 1 in. is left between the ends of the adjacent girders, which are bolted through slotted holes. The top flanges of the girders are reinforced by countersunk-riveted 12-in. channels, flanges down, which overlap the sliding end at expansion joints and are bolted to the girder through slotted holes.

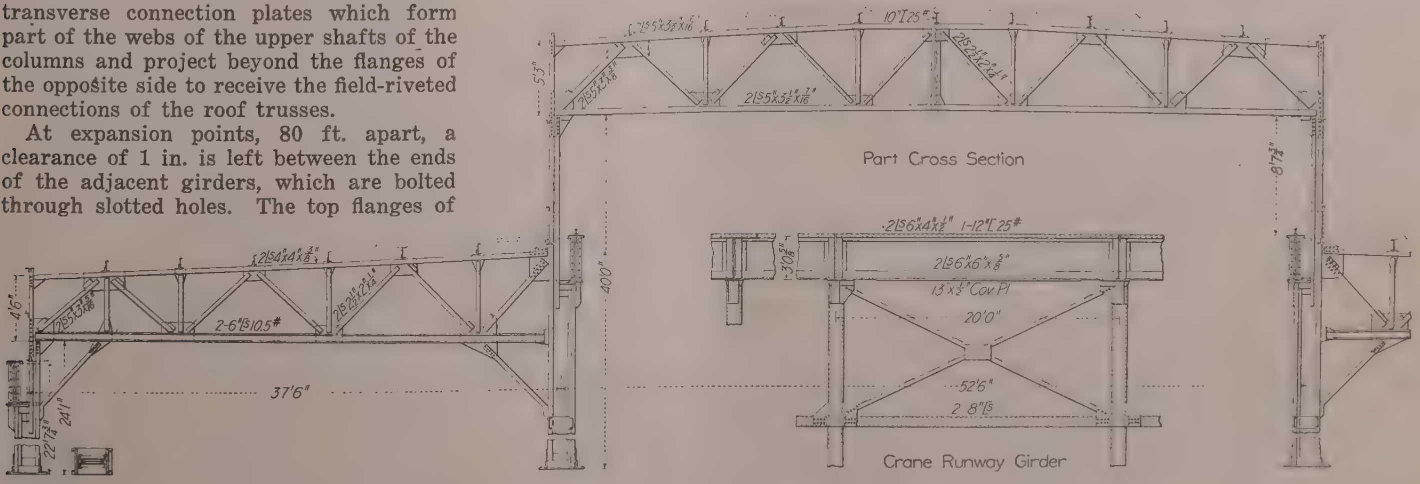
The 60-lb. track rails are seated without cushions on the backs of the flange channels, and are secured by pairs of hook-bolts with the bent ends passing through holes in the webs of the channels to engage the top flange angles and having nuts on the upper end which take bearing on the rail webs.

The longitudinal panels between the columns below the runway girders are X-braced by single angles field-riveted to connection plates shop-riveted across the faces of the columns, the lower plates serving also as jaws for horizontal struts made of a pair of 8-in. channels with their top and bottom flanges latticed.

As there are no wall columns at the gable ends of the building, the runway girders are supported there on special columns made of pairs of 12-in. latticed channels, braced at their upper ends by light transverse lattice girders 4 ft. deep set close to



Details of Longitudinal Bracing in Car Repair Shop Framework



Elevation of Main Transverse Bent; Typical Roof Trusses and Columns in Car Repair Shop the face of the brick wall and made of single 12-in. channels for the flanges and single 2½ x 2½ x 5/16-in. pairs of diagonal angles riveted between connection plates on the channel flanges.

Mezzanine Floors

In nine panels of the 37½-ft. aisle there is a second floor with a reinforced-concrete slab carried on the top flanges of 12-in. longitudinal I-beams, which are about 4 ft. apart and are connected by hitch angles to the webs of 15-in. transverse I-beams in the main transverse bents. The tops of the I-beams are 7 ft. 10 in. below the bottom chords of the roof trusses and are supported at two intermediate points from panel points of the roof trusses by vertical suspenders made of pairs of 3½ x 3½-in. angles.

A wooden mule traveler running from end to end of the site on a broad-gage track laid in one of the intermediate aisles was employed to erect the steel work. It had two long booms operated by a hoisting engine on the traveler, and erected the complete trusses and columns as it receded from them.

Paint Shop Having Large Steel Frame Windows Separated by Brick Piers

Planing Mill and Lumber Storage

The 81 x 300-ft. planing mill and 81 x 200-ft. storage building for finished lumber have similar steel frames with transverse bents consisting of a full-span roof truss 8 ft. deep with monitor and two vertical wall columns. Light longitudinal latticed trusses 6 ft. deep at the tops of the columns, latticed trusses along the sides of the monitor and eleven lines of channel and I-beam purlins seated on the top flanges of the roof trusses connect the bents. Light longitudinal lattice girder trusses 7 ft. deep connect the roof trusses on the center lines in alternate panels.

The trusses in the main and monitor roofs are X-braced with 1-in. and ¾-in. clevis rods in the planes of the top chords. The bottom chords are made with pairs of channels riveted together back to back and intended to receive trolley hoists or carry other light loads at any point; all other members are made of pairs of angles back to back. The end lower-chord gusset plates project below the bottom flanges and are riveted between the flange angles of the wall columns, of which they form the upper section of the web plates.

The inner flanges of the I-shaped wall columns terminate at the bottom chords of the roof trusses and the outer flanges are made continuous to the top chords of the trusses, thus forming an end vertical post integral with the columns and also serving as a member of the longitudinal eave truss, which is continuous from end to end of the building except for expansion joints 80 ft. apart. At such joints the longitudinal trusses and all purlins are connected by bolts through slotted holes.

The planing-mill frame has transverse bents about 80½ ft. wide and 20 ft. apart, connected at eaves and centers by longitudinal lattice girders as deep as the roof trusses. Every second panel is X-braced by diagonal screw rods in the planes of the top chords. The roof is lighted by a full-length monitor 23 ft. wide with clerestory windows.

Paint Shop

The 53 x 200-ft. paint shop has brick and glass walls and no columns. The side walls are virtually series of thin wide piers seated on a continuous concrete foundation wall extending from a little above floor level to the original surface of the ground. The wide steel window frames were erected on the concrete wall before the brick wall was begun.

Transverse trusses 20 ft. apart support the roof. These are similar to those in the planing mill building except for dimensions, bearings and the absence of a monitor. The bottom chord is made of a pair of 4 x 4-in. angles full length. The top-chord angles are also full length, slightly bent at the center to provide for the roof pitch. The eaves trusses are field-riveted across the ends of the roof trusses with which their bottom flanges are flush, and ¾-in. sole plates riveted across the bottom flanges of both trusses take bearing across the full width of the top of the brick wall.

Bracing for the trusses is supplied by eight lines of 10-in. purlin channels seated on the top chords, by a lattice strut 24 in. deep on the center line, by knee-braces to the bottom chords of the roof trusses in each alternate panel, and in four intermediate panels by X-brace rods with clevis connections in the planes of the top chords. At the center of the building and at the second panel point from each end every longitudinal member has expansion joints with bolts through slotted holes.

The sand-blast house has a steel framework, brick gable walls and concrete side walls reaching up only about 4 ft. above floor level, all the space above them being glazed. Cars are run into the building and the old paint removed by the sand blast. No crane service is provided for it.

The buildings were designed by the engineering department of the Central Railroad Company of New Jersey, of which Joseph Osgood is chief engineer. The structural steel was fabricated and erected by The Phoenix Iron Company.

Explosions in Sewers

Results of an Investigation by H. J. Kellogg, Assistant City Engineer of New Haven, Connecticut

For the past year the Department of Public Works of New Haven, Conn., has been investigating causes of sewer explosions and endeavoring to discover means of preventing them. The studies were not confined to New Haven alone, but included the experience of the principal cities of this country and Canada. The recent sewer explosions in Pittsburgh and Detroit have impressed upon engineers the importance of preventing accidents of this sort and the paper on sewer explosion presented at the annual meeting of the Connecticut Society of Civil Engineers on Feb. 11 by H. J. Kellogg, assistant city engineer of New Haven, is particularly timely. An abstract of the paper follows.

The first serious explosion in New Haven occurred twenty years ago in a 54-in. circular brick sewer when four men were inside cleaning it. The explosion appeared to the men as a great ball of fire coming like a bolt of lightning from the upper end of the tube. All of them were knocked down and temporarily stunned. One effect of the blast was to blow the water out of the sewer and check the flow. It is believed that the explosion was caused either by illuminating gas or by waste products from a dyeing and cleaning shop. The men in the sewer carried lanterns, which ignited the combustible vapors.

Danger of Lanterns

Another blow-up occurred last winter in a 42-in. egg-shaped brick sewer while men were removing deposit from its interior. They were burned about the face and hands. Although no positive evidence was available, it is thought that the explosion was due to benzine, naphtha or gasoline used by a concern where lithographic work was done. After this second explosion the men were forbidden to carry lanterns within the sewer and a dry-battery electric light was substituted.

In 1886 50 ft. of the arch of a 60-ft. circular brick sewer were blown off by an internal explosion. In 1894, during a terrific thunderstorm, a 66-in. horseshoe section brick sewer was split open for a distance of 400 ft. The gap in the arch at its widest part was 1 ft. The crown of the sewer was 12 in. thick and the brickwork was in three wings. It is possible that this sewer may have been struck by lightning.

Telephone Conduits

There have been a number of explosions in telephone conduits. In Bridgeport, two years ago, a manhole cover was blown off by ignited illuminating gas and the gas kept burning for hours before the unmetered supply could be stopped. Four years ago, in the same city, an explosion in a conduit manhole broke forty panes of glass in adjacent buildings. In Fairfield sparks from a trolley car set off a blast that blew up 675 ft. of ducts. Ventilated manhole covers, Mr. Kellogg states, have checked this kind of conduit trouble, but a bad break in a gas main would threaten the underground system.

In Columbus, Ohio, according to a letter from C. J. Lauer, chief of the fire department, no explosions in sewers have occurred in fifteen years. There have been two explosions in underground-telegraph ducts caused by accumulation of natural gas. The gas was ignited by a fire engine passing over the manhole.

The worst sewer explosion on record occurred in Pittsburgh on Nov. 25. More than a mile of 8-ft. sewer was shattered and the cost of repairs and damage is estimated at half a million dollars.

Hydrocarbon Oil

In Buffalo, N. Y., according to George H. Norton, deputy engineer commissioner, there have been serious complaints from gas odor in certain sewers for three years past at periods following thawing weather after frost and heavy snow. This has been traced to the use by the railroads of what is called a hydrocarbon oil, which is used by them to prevent freezing of their switches and interlocking places in passenger-station yards and at other congested points. At low temperatures this oil crystallizes; the crystals are soluble in water and are thus carried into the sewers. The gases, it is stated, are highly explosive and could readily be the cause of a serious explosion.

Roscoe N. Clark, city engineer of Hartford, reports two serious sewer explosions in 1913. The first lifted one manhole head and several covers and shattered windows in adjoining houses. The second damaged the new east-side pumping station. Both of the explosions took place in the vicinity of the works of the Hartford City Gas Light Company. To protect the pumping station there is now being built a ventilating stack with a by-path equipped with an electric fan so that artificial ventilation can be resorted to if the natural ventilation proves insufficient.

Washington Ordinance

The following ordinance regulating the discharge of inflammable matter into sewers is in force in Washington:

"No person shall make or maintain any connection with any public sewer or appurtenance thereof whereby there may be conveyed into the same any hot, suffocating, corrosive, inflammable or explosive liquid, gas, vapor, substance or material of any kind. And no person shall cause to enter or flow into any public sewer or appurtenance thereof any hot, corrosive, suffocating, inflammable or explosive liquid, gas, vapor, substance or material of any kind, provided that the provisions of this paragraph shall not apply to water from ordinary hot-water boilers of residences." There are rules in Washington for the installation of garage traps.

Oil Separators in New York

Prior to the regulation requiring the installation of oil separators in garages in New York, according to Robert Adamson, fire commissioner, the larger number of sewer explosions were limited to what is known as the "garage zone." Since their installation the number of explosions have been greatly reduced. Since June 5, 1912, there have been eleven sewer explosions in New York.

Illuminating gases can be kept out of the sewers more easily than out of the conduits for wires, as the latter lie nearer the surface and afford better channels for the escape of the gas that may accumulate under the nearly gas-tight pavement. In New Bedford it is intended to have the ducts calked with oakum and some impervious substance like tallow that can easily be taken out. Greater care is now generally taken in laying gas mains and calking joints.

Perforated Manhole Covers

Constant ventilation to rid the sewers and conduits of these dangerous gases appears to be good practice, especially to expel the illuminating gas. In Springfield, Mass., some of the catchbasins are trapped. Perforated manhole covers are in use in Pittsburgh. Some years ago in New Haven perforated manhole covers were tried, but the protests were so vigorous that the holes were plugged with bolts and now all covers are solid plates. There is ventilation through manholes, but it is incidental to frequent removal of covers.

The objection against allowing sewer air to escape into the street on the ground of the danger of infection is without good cause, according to Mr. Kellogg. In twenty-five years' connection with sewerage work he has never heard of a sewer workman contracting typhoid fever or any other disease that could be attributed to so-called "sewer air."

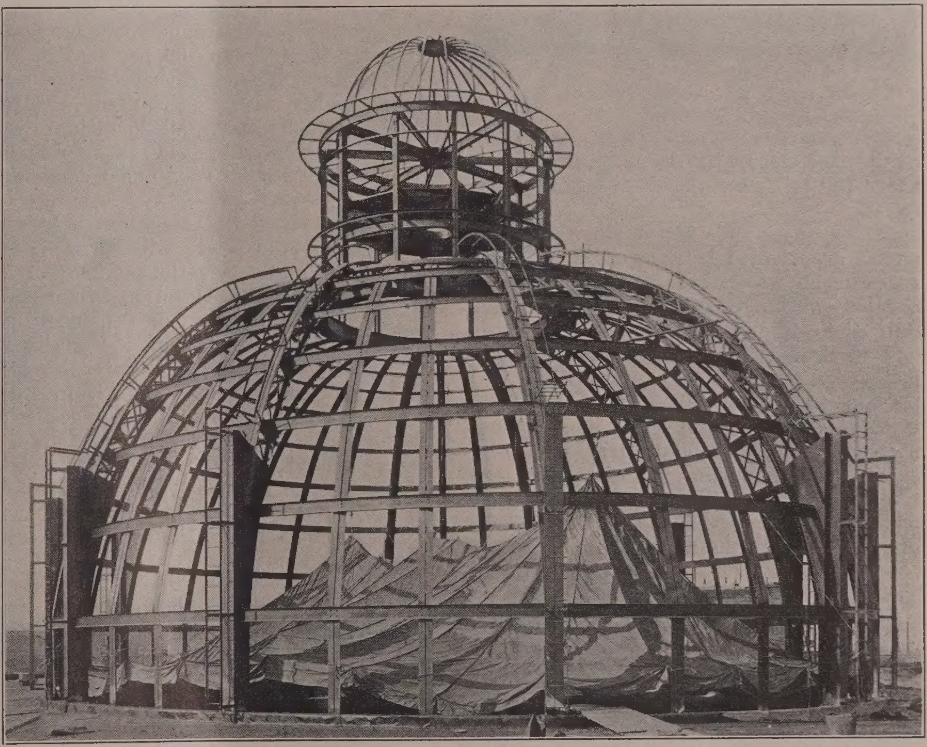
An Employment Bureau has been established by the Boston Society of Civil Engineers for the benefit of its membership. The bureau is intended to be a medium for securing positions for its members and applicants for membership, and also for furnishing employees to members and others desiring men capable of filling responsible positions. Two lists are kept on file at the society rooms, one of positions available and the other of men available, giving in each case detailed information.

Dome of the Kahn Building in San Francisco

95 x 72-Foot Skylight Dome and 22-Foot Lantern with Steel Ribs Supported on Five-Story Columns

A STEEL dome of very large dimensions has just been constructed for a department-store building in Oakland, Cal. It provides an exceptionally great skylight area, has special details of construction eliminating heavy drums and girders often used in such structures, and is notable in that the surface although symmetrical is not one of revolution.

At the base the plan is a 72 x 95-ft. ellipse, which gradually changes as the elevation increases, until at the foot of the lantern it becomes a concentric 22-ft. circle. The resulting dome surface is therefore not a true ellipsoid, although its elements are symmetrical curves. The main framework consists of light latticed radial girders which, instead of being seated on girders or connected to a massive drum, have rigid web connections to columns braced at about the same elevation to the horizontal members of the general floor or roof construction.

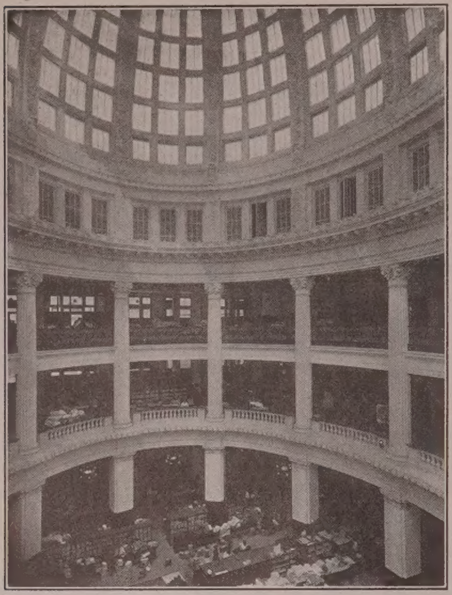


Steel Framework for Elliptical Dome and Circular Lantern

The dome surmounts a rotunda extending to the top of the building through full-size elliptical openings in each of the four floors above the ground. It has a height of 36½ ft. from springing line to the lantern, which is 22 ft. in diameter and 15 ft. 3 in. high, surmounted by a circular dome with a radius of 9 ft. 8 in.

Ribs and Columns

The twelve main dome ribs are 18 in. deep at the crown and 37 in. deep at the haunches. They have I-shape cross-sections made with two pairs of 5 x 3½ x ⅜-in. flange angles, which are latticed, except below the haunches, where the lattice is replaced by solid web plates integral with the upper parts of the supporting columns. These columns project several feet above



Rotunda and Finished Dome of Kahn Building

their connections with the dome, to provide, above the roof, for the erection of future stories.

At the upper ends of the ribs the flange angles are riveted between pairs of horizontal bent connection plates on the top and bottom flanges of two 18-in. channels, latticed together back to back to form the lantern drum. In each panel between the main ribs there are two intermediate ribs made of pairs of 5 x 3½ x 5/16-in. angles riveted back to back. The ribs are connected by ten horizontal tiers of 10-in. curved channel purlins. The purlins are connected by hitch angles to short web plates in the dome ribs, and the purlin webs are parallel with the flanges of the ribs.

Lantern Framework

The lantern framework has five vertical posts made of pairs of 8-in. channels back to back, with their upper ends connected by a circular 10-in. channel which supports the ribs of the small dome, four of which are made of pairs of 10-in. channels, the eight intermediate ones being made of single channels. The small dome is almost hemispherical, and the upper ends of its ribs are riveted to a ⅜-in. center plate.

For architectural considerations the steelwork of the main dome is covered with sheet metal and the panels between the members are glazed. The sides of the lantern have ventilating louvres and its dome is covered with sheet metal. C. W. Dickey was the architect for the Kahn building and Morris C. Couchot, consulting engineer.

Letters to the Editor

Surge Tank at San Francisquito Power Station No. 1

Sir: Having read with interest the article published in the Engineering Record of Sept. 13, 1913, page 299, regarding the surge tank at San Francisquito power station No. 1, and later the discussion in letters by R. D. Johnson and Prof. W. F. Durand, I have made some calculations as to the behavior of the tank in question under various conditions.

The conditions imposed, to quote from Prof. Durand's letter in the issue of the Engineering Record of Dec. 27, 1913, page 733, follow: "Given a reservoir level anywhere in its 75 ft. of range with a resultant possible upper conduit velocity varying from 10 ft. per second for higher levels down to 7 ft. per second for the lowest level—with a possible sudden change varying from as much as 100 per cent of the lower rates of flow to perhaps 60 per cent of the higher rates: Required, a surge chamber which, under all conditions, will provide substantially dead-beat movement of the water level, at least under demanded load."

Assuming that the reservoir is at its lowest stage, the water will stand in the tank at the static elevation of 2961 or 75 ft. below high-water elevation. Suppose a sudden 100-per cent load is demanded, requiring a draft velocity of 7 conduit-feet per second, which, as the water level in the tank drops, increases slightly in order to hold the power constant. The action of the surge tank under these conditions has been computed, second by second, by means of arithmetical integration and the results are plotted graphically in Fig. 1. It will be noticed that before the conduit velocity has been brought up to 3½ ft. per second, or half that required, the surge tank is completely empty and the penstocks start to draw air. Instead of obtaining "substantially dead-beat movement of the water level," impossible operating conditions actually result. In other words, the surge tank apparently does not nearly fulfil the requirement stated as a premise for the design.

A case not nearly so severe is shown graphically in Fig. 2. Here the reservoir is still at its lowest stage and the conduit flowing 5 ft. per second. A load change of approximately 2 per cent is demanded, necessitating a draft of 7 conduit-feet per second. To hold the power constant, this draft, by governor action, increases as the water level falls until at the quarter cycle it reaches 7.21 ft. per second. The net head on the wheels at the start was figured as 847 ft. and the water level in the tank at El. 2942 (low water minus 19 ft. lost in friction for 5 ft. per second velocity).

At the end of a quarter cycle, at which time the conduit velocity has been brought up equal to the draft velocity, the water level in the tank has dropped 24.7 ft., while the gradient has dropped only 20.2 ft. The surge or over-reach is therefore 4.5 ft., or 22 per cent greater than the gradient change. In order to obtain dead-beat movement of the water even with this small load change the reservoir must be at least 20 ft. above its lowest stage. At this point the movement of the water level in the surge tank occurs 20 ft. higher in the conical section, where at the start the area is 60 per cent greater. For stages of the reservoir at which the movement of the water takes place within the cylindrical section the tank is ample to assure dead-beat conditions with large changes of load.

The conclusions derived from these cases and several other computed combinations of conditions seem to point to the fact that the adoption of the conical shape for the lower part of the surge tank is detrimental to its action as a regulator, chiefly because the static level of the storage pond is subject to large variation. By allowing a surge of 6 or 7 ft., instead of striving for dead-beat conditions, a great saving in tank area could have been made, and since a 7-ft. surge on a net head of 850 ft. amounts to only 0.83 per cent, it is difficult to see how its effect could be noticed even with exacting governing conditions.

Mr. Johnson's statement in regard to the 30-ft. differential regulator would indicate that for conditions as shown in Fig. 2 the surge would be negligibly small as compared with the operating head.

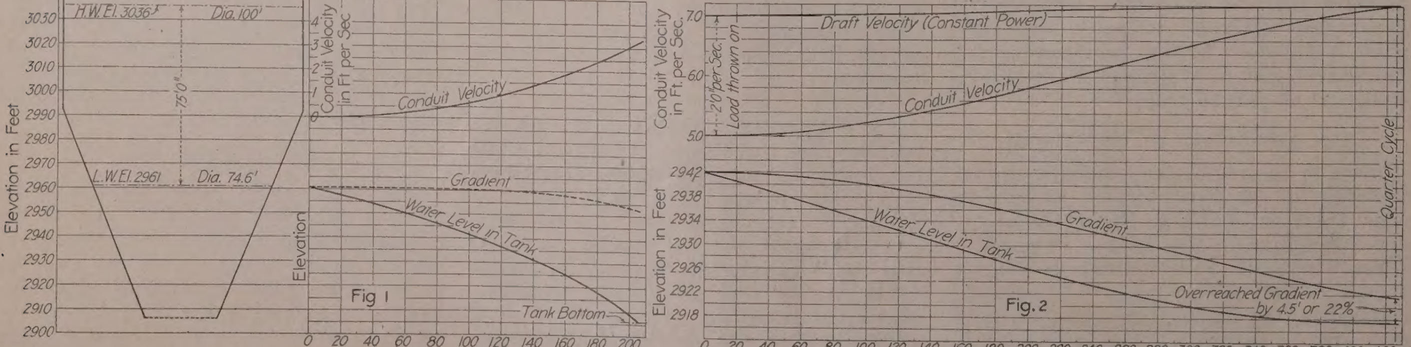
Roy Taylor.

Niagara Falls, N. Y.

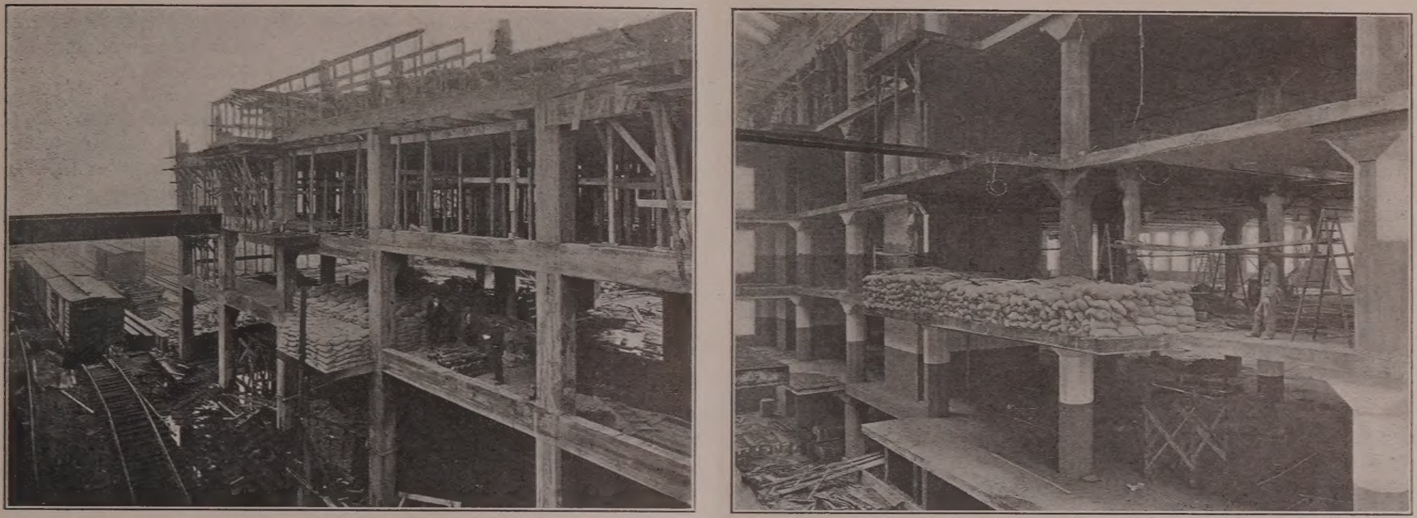
[The above letter was referred to Prof. W. F. Durand for a reply, which follows.—Editor.]

Sir: Referring to the communication of Roy Taylor, I would reply as follows:

It is evident that if we could all agree to define fully the terms employed there would be little ground left for discussion or controversy. In using the term "substantially dead-beat movement" I did not mean to imply absolute dead beat. In comparison with a total head of some 900 ft., a surge of 3 or 4 or even 5 or 6 ft. and occupying a period of several minutes, was considered as furnishing a substantially dead-beat movement. Surges of this order, and as indicated in Mr. Taylor's second case, are therefore anticipated with the reservoir



Figs. 1 and 2, Showing Condition under Load Variations of 100 and 29 Per Cent Respectively for San Francisquito Surge Tank



Detroit Building with Panels of 20-Foot Span (Left) and Chicago Building with Panels of 28-Foot Span (Right) for Ford Motor Company

at the lowest level and are included in the expected program of operation.

In the effort to be brief, in my letter published in the issue of Dec. 27, 1913, it appears that I have unintentionally misled Mr. Taylor and perhaps others as to the exact program of operating conditions under which it is expected that the chamber will provide substantially dead-beat operation. While it is true that the chamber was investigated for sudden demands represented by increments of velocity varying all the way from 0.75 ft. per second to 6 and even 8 ft. per second, these latter extreme conditions were not imposed on the chamber with the Fairmont reservoir at its lowest level. Many cases were examined for this lower level, including surges singly and in series, and covering the results found by Mr. Taylor in his cases. As an operating condition, however, the maximum sudden demand was taken at about 40 per cent of the maximum possible load, giving a demand velocity of about 2.7 ft. per second. Under this condition moderate surges, as found by Mr. Taylor, are to be expected and are considered as acceptable for the range of conditions anticipated.

First and last some thirty or forty cases were worked through by the process of numerical integration, presumably the same in general character as used by Mr. Taylor, and the results were thus developed for a wide range of demanded loads, as noted above, and covering such combinations of operating conditions as may be reasonably anticipated in practice. They do not, however, include the extreme condition assumed by Mr. Taylor in his first case, and I am glad of the opportunity to state more fully the range actually included.

From a somewhat different point of view the form and dimensions of the chamber may be said to have developed as follows: (a) For the most probable average reservoir level a chamber was developed giving closely dead-beat movement over a wide range of operating conditions. (b) For structural and local reasons it was desirable to make the chamber tapering toward the lower end, primarily on account of the proper care of the hydrostatic pressures developed with a full chamber and secondarily for economy of construction. (c) This form will obviously introduce some limitations when operating at the lower or lowest reservoir levels. (d) Such limitations at the lower levels were carefully and fully developed and accepted as defining a field of satisfactory operation sufficiently wide to meet all reasonable or probable demands.

Stanford University, W. F. DURAND.
California.

Progress in Electric-Power Transmission

SIR: Referring to the editorial in the Engineering Record of January 10, 1914, page 33, entitled "Progress in Electric-Power Transmission," we would call your attention to a criticism of the use of synchronous condensers which is evidently based on an erroneous view of their function in that plant. The condensers are not installed to compensate for the capacity effects of the line, as stated in the editorial, and any use of this kind made of them is incidental. The condensers are used because without them the cost of the line would have been greatly increased to give the degree of regulation absolutely necessary for proper commercial service. We would especially point out that the line capacity is not an important factor as regards regulation, and that the condensers would still be necessary had the line no electrostatic capacity. That is to say, the condensers are included as an integral part of the design, because economy has been obtained in the design as a whole through their use.

You also say: "On these very long lines at high voltage the electrostatic capacity effect is so considerable that inductive load, cordially abominated by the average central station, becomes a welcome friend. Many of the long Western lines have gladly taken on an unusual amount of induction motor load to compensate the line capacity, and in this Big Creek development two large synchronous machines are deliberately installed in the main substation for the same purpose." We have mentioned above the error in regard to the use of the synchronous machines, but would point out in addition, first, that the power is actually to be delivered over this line to "an unusual amount of induction-motor load," which should bring this plant into the class of "many of the long Western lines" which are presumably benefited by such load; second, that the balancing of line capacity by induction-motor load is of little, if any, practical benefit to the transmission system; third, that, in fact, induction-motor load is a great detriment to the regulation of the transmission system.

DWIGHT P. ROBINSON,
President and General Manager, Stone & Webster Engineering Corporation.
Boston, Mass.

[The use of synchronous machines for line regulation is perfectly well known and it is one of the reasons for the common Continental practice of using synchronous-motor generators instead of ordinary rotaries. In the editorial of Jan. 10 it was assumed that these condensers were installed for power-factor improvement, which is ordinarily the object of these machines, but Mr. Robinson indicates that this is not the case in the Big Creek installation. There these synchronous condensers were installed to regulate the voltage. It would, therefore, seem that a subordinate function has been mistaken for a principal one in the matter mentioned by Mr. Robinson. Considering that the Big Creek installation is the first in which these machines have been installed for the primary purpose of voltage regulation—the other advantages as power-factor improvement, etc., being secondary—the operating results will be watched with interest. Regarding the point on induction motor load Mr. Robinson's position seems well taken.—EDITOR.]

Concrete Buildings for Motor Company

SIR: In your issue of Jan. 31 you described and illustrated the reinforced-concrete roof trusses and cantilevered balconies in the Ford Motor Company's assembly plant at Chicago, the structural features of which were designed by the Condron Company. In your issue of Feb. 14 you described and illustrated the addition to the Ford Motor Company's factory at Detroit, and in this connection used both on your front cover and in the text reproductions of the photograph taken of the Chicago building, which would leave your readers with the impression that the concrete construction of the two buildings was the same.

There are several fundamental differences between the designs for these two buildings, and we think that perhaps it will be interesting to your readers to note the difference in the construction of the floors and cantilevered balconies. One of the accompanying photographs shows the Detroit building with panels of 20-ft. span and cantilevered balconies each about 10 ft. long with an overhang of 7 ft. beyond the center line of columns. The other photograph is of the Chicago building, with panels of 28-ft. span and cantilevered balconies varying in length from 14 to 56 ft., with an overhang of 8 ft. 6 in. beyond the center lines of columns. Note the absence of deep spandrel girders in the Chicago building and the fact that this makes construction work more simple and offers no obstruction to light. The photographs show test loads on the balconies.

CONDRON COMPANY,
By T. L. Condron.
Chicago.

Ohio Conservancy Act

SIR: An editorial in your issue of Feb. 7, page 149, criticised the recently enacted Conservancy Act of Ohio in that it does not provide for a conservancy commission to have control of all work in the State in the same manner as public-service commissions. The question of providing such a commission received very careful consideration in the drafting of this bill, and we believe we were right in not making provision for a State-wide commission at the present time for the following reasons:

Numerous commissions have been provided for in the State of Ohio in recent years and there is a very marked sentiment against the creation of any additional organizations of this character. Had provision for a State-wide commission been included in the bill, the entire measure might have been defeated and actual work for flood prevention delayed indefinitely. This considered alone would have been sufficient to determine the question.

In the second place, it is the experience of the Central States that a real demand for a State-wide commission of this sort must precede its creation, if it is to be an efficient organization. In one of the Central States where a State-wide organization was created recently a country newspaperman who had been friendly to the administration was placed in charge and is now acting as consulting engineer for various local organizations over the State. In another case a law was passed requiring all work of this character to be approved by a State official. The man chosen to fill the position was entirely without experience in this class of work. He has been and is now approving plans which are almost completely without engineering merit and which must fail in operation. His approval of these plans is giving a false sense of security concerning some very inefficient work.

In another State a commission has charge of a certain portion of the reclamation work within the State, but the work done under the control of this commission has been generally of a poorer character than that carried out without such State supervision. In another State where work of this character must be supervised by State officials not a single member of the board having control of developments of this character has had any extensive experience in this class of engineering, except as he has acquired it as a member of such board, and the findings of this board furnish very clear evidence of this lack of experience. In another State where State control has been aimed at the engineering work throughout the State is in the control of an officer who is not an engineer, and who has had no experience in engineering. In some States there are very efficient commissions, or State officials, having supervision over power developments or irrigation developments. We believe that in every case where such officials or commissions are thoroughly efficient they were not provided until the need for their services was generally recognized.

As a matter of fact during the last twenty years many millions of dollars have been spent in the Central States in the reclamation of wet and overflowed lands and in the prevention of floods. By far the larger part of this work has been done without any effective State control. The tendency for various projects to interfere with one another is not so great as has been sometimes assumed by writers on the subject, and while a poor character of engineering has been quite common on many of these projects there has not, on the whole, been serious loss or lack in efficiency because of the failure of suitable co-operation.

You make the statement further that the courts exercise the only control over conservancy districts which is provided in the law. The governing body of a conservancy district is a board of directors which has charge of the developments of the district, except where legal processes are necessary. Most of the jurisdiction conferred upon the courts is necessarily placed there in order to cover constitutional requirements for the protection of property rights, and no State commission could be organized with the right to take away such jurisdiction.

It may not be generally realized that in the State of Ohio the numerous rivers have practically no connection with one another, and that on the Miami River, the Mahoning, the Scioto, or on other rivers any improvements undertaken will have no effect on other rivers, except indirectly upon the Ohio. In the case of the Great Miami River it is proposed to organize the entire river valley, from its source to its mouth, into a single conservancy district, and there can be little possibility of any conflict between this district and others which may be organized.

The framers of this bill realize very keenly the advantages of efficient State-wide supervision and control. However, in view of the fact that such a provision at the present time would perhaps have defeated the entire measure and that where the creation of such State-wide commissions has been in advance of the well-recognized need for their existence that they have not been notably efficient, and in view of the further fact that the progress of flood prevention and of the conservation of the waters of the State can at present proceed effectively without such a commission, it seemed best to leave such a provision for the future. It is the hope of the framers of this bill that by the time it has been in operation a few years it will be possible to so amend it as to provide for efficient State-wide supervision.

There are about twenty States which now have laws covering somewhat the same ground as the Conservancy Act of Ohio. If the writer of your editorial had been acquainted with these various laws we believe his principal impression upon reading the Conservancy Act of Ohio would not have been the lack of a State commission—a lack which is common to similar laws in most other States—but it would have been that here, for the first time, there is an orderly and complete codification of the important features of a reclamation and flood-prevention law; that here for the first time, in the Central States at least, provision is made for definitely locating responsibility for the control of streams and the prevention of their misuse, and that this act is also an innovation, in that it secures to the public the control and income of water supply created by the public. It is difficult to estimate accurately the significance of this law, unless one makes a comparison of it with the laws of a similar nature in other States.

MORGAN ENGINEERING COMPANY,
Arthur E. Morgan, President.
Memphis, Tenn.

Raid on Stock-Selling Engineering Company

SIR: In the Engineering Record of Feb. 14 I find an article in the Current News section entitled "Raid on Stock-Selling Engineering Company," and an editorial comment, which afforded me considerable amusement, and more satisfaction because the president of the United Engineering & Construction Company, William R. Rose, and his worthy partner, Theodore Rodmann, and others whom I met did their utmost to enlist my services.

I went to Cleveland in good faith, believing the enterprise legitimate, and during an interview with Mr. Rose I was led to believe that his company was just as represented, all of which was backed up by a full-fledged working force, and, to all appearances, the field was decidedly attractive, except for one thing—they insisted upon an investment of $1000. I made them a counter proposition which they took under advisement, and the following day informed me that on account of the company being absolutely co-operative they could not accept my offer.

Subsequently they compromised to the extent of letting me in on a payment of $500, balance to be paid from my earnings. To make a long story short, I didn't "bite," but, on the contrary, was satisfied the whole thing was out of order, and I took the trouble to write a friend in Cleveland and suggested that the proper authorities should investigate the company and expose the idiotic scheme. I also informed people in Buffalo to be on the lookout, as they were establishing an office in Buffalo at or about the same time. In both instances my object was to do what I could to put a stop to such schemes, which are a great injury and a discredit to the engineering profession.

RICHARD H. BRODHEAD,
Engineer and Contractor.
Greenville, Pa.

[This is the company which endeavored to insert advertisements in the Engineering Record. It was investigated by our Searchlight Section and reported to the Postal authorities in Washington, D. C.—EDITOR.]

EXPORTING TWISTED STEEL REINFORCEMENT BARS to the Pacific Coast by the Panama Canal from England and the Continent was discussed in a recent report from the U. S. consul at Liverpool. This report quoted square or round bars at $22.14 and $29.19 f.o.b. at Antwerp and Liverpool respectively, plus cost of twisting, thus making it evident that there is no prospect for exporting English steel to the Pacific Coast at present. The report also states that although there are in the Liverpool district two large manufacturers producing reinforcement bars, most of the bars used in England are imported from the Continent.